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CONTENTS

July, 1957

Papers

Number

Research and New Developments in Photogrammetry
by Charles L. Miller 1305

Foreword to Manual on Highway and Bridge Surveys and
Chapter I - State Plane Coordinates
Progress Report of the Committee on Highway and Bridge
Surveys, Surveying and Mapping Division 1306

New Highways and Photogrammetry
by John G. Ladd 1307

Discussion 1310

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Journal of the
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RESEARCH AND NEW DEVELOPMENTS IN PHOTOGRAMMETRY*

Charles L. Miller[1]
(Proc. Paper 1305)

INTRODUCTION

Photogrammetry is one of the most dynamic technological fields contributing to the technical and professional advancement of civil engineering. The purpose of this paper is to report to the members of the American Society of Civil Engineers on the significant advances in photogrammetry during the past five years and to discuss some of the current research of importance to civil engineers. Civil engineers can no longer consider photogrammetry a novelty of minor importance. The technical and professional advancements of photogrammetry are having a profound impact on several areas of civil engineering. Although most of the research and new developments reviewed in this paper have been reported in the photogrammetric literature, very little has been published in the civil engineering literature and may therefore have escaped the attention of many members of the American Society of Civil Engineers.

Research and development in most technological fields is usually motivated by the three goals of

1) higher precision
2) time and cost reduction
3) new applications.

Underlying all three goals are economic considerations which must justify the research and development. The distinction between research and development has been defined in many ways. For our purposes, we may say that research is extending our bounds of knowledge regarding a given subject or problem, and development is the perfection of a new method, technique, or instrument for the useful application of our new knowledge.

Photogrammetry, by definition, is the applied science of using photographs for making physical measurements. Since all branches of science and

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engineering are concerned with making physical measurements, photogrammetry has an actual or potential application throughout the entire spectrum of scientific and engineering endeavor. In addition, photogrammetry draws on the technical knowledge and developments of many branches of science and engineering. Due to these broad relationships of contribution and support, photogrammetric research and development cannot be segregated into its own little compartment. It is not an isolated and specialized field but one completely integrated in the complex of scientific and technical endeavor. This is true of many subjects of interest to civil engineers. In this paper, we shall be principally concerned with photogrammetry as related to civil engineering with emphasis on the area of surveying and mapping.

Advances in Precision

General

With respect to the research and development goal of higher precision, photogrammetry continues to advance. Higher precision involves research in the area of more knowledge about our physical systems. Essentially we are concerned with research in error analysis—the source and magnitude of our errors. Development is concerned with the practical and economic perfection of methods and instrumentation which will reduce the errors we have learned about through research.

Precision Photography

Since the entire foundation of our photogrammetric system is the photograph, the precision of our raw data—photographic images—occupies considerable attention in research and development. The end product of such activity is better lenses, cameras, and photographic materials. By better we mean the twofold goal of (1) higher image quality (as judged by definition, sharpness brightness, resolving power and similar criteria) and (2) higher standards of geometric relationships (as judged by distortion, deformation, and stability). In each case these goals must be strived for within the limits of economic feasibility. A new system is not an engineering success if its cost prohibits useful application. For this reason a scientific success is not necessarily an engineering success.

As a result of research on the components of the precision photograph, the following developments are given as examples:

a) New precision aerial photographic lenses such as the Zeiss Pleogon, Zeiss Topar, Wild Aviotar, Wild Aviogon, and the Bausch and Lomb Planigon.

b) New precision mapping cameras such as the Fairchild T-11, Zeiss RMK series, Wild RC series, and the Hycon Cartographic camera.

c) Advances in camera calibration knowledge and ability through the activities of such organizations as the National Bureau of Standards, U.S. Geological Survey, and the Fairchild Camera and Instrument Corporation.

d) Advancements in image motion compensation systems and camera stabilization systems such as the activities of Hycon, Fairchild, Aeroflex, and Boston University.

e) The LogEtronic system of automatic dodging for higher print quality.

f) Advances in dimensional stability of photographic firm base such as the new duPont Cronar.

Convergent Photography

One significant trend in United States photogrammetric practice has been the renewed interest in convergent photography as a means of increasing our precision capabilities. This trend has been led by the research and development activities of the U.S. Geological Survey. The Engineer Research and Development Laboratories have also been very active in this area of research. The advantages and disadvantages of convergent photography for large scale—small contour interval mapping such as required for civil engineering projects has not been fully evaluated and is a subject of future research.

Instrumentation

Closely associated with research and development on the photographic link in the photogrammetric system is the entire area of instrumentation for the evaluation, measurement, and reduction of the raw data captured by the photograph. If our treatment of the photographic data is to be analytical, we are interested in direct linear measurement on the photographs to a high order of precision. Research and development on new methods of making such measurements to higher and higher levels of precision is a subject of considerable activity. Development of new and better comparators is an active problem with such firms as Wild, Zeiss, and David Mann.

If we are going to use our photographs for the creation of an analog or model of the object photographed, our instrumentation for performing this function must keep pace with our goal of higher precision. The first order stereoplotter, which reached an extremely high level of perfection many years ago continues to improve. The new models of the Zeiss Stereoplanigraph and Wild Autographs are now in use in the United States. In addition, Abrams is now operating the Santoni first order plotters in the United States. The Nistri first order plotters are now being introduced to United States practice by the OMI Corporation of America and no doubt will play an important role in the future.

The relationship of economy to precision plays an important role with respect to stereoplotters. However, we must be careful in comparing the cost of a first order European plotter with one of our double projection American plotters. A cost ratio of ten to one does not indicate a similar precision ratio although some civil engineers have gained this mistaken conception. A large portion of the differential is caused by the difference in cost of building a universal instrument versus a specialized instrument.

With respect to the results of research and development of new types and improved versions of stereoplotters in the United States, the new Bausch and Lomb Balplex is a good example. The Balplex is an outgrowth of the Twinplex plotter and the ER-55 projecter developments from the research laboratories of the U.S. Geological Survey. In the Balplex, our goal of higher precision has been obtained; a low economic cost making precision photogrammetric instrumentation available on a much wider basis than would be otherwise possible. Through the availability of such low cost precision plotters as the Balplex and Kelsh, we are seeing the introduction of photogrammetric divisions in a large number of our small and medium size civil engineering and surveying firms.

Time and Cost Reductions

Precision Versus Economy

The research and development goals of lower cost and faster time provide strong economic impetus for improvement. In fact, in many cases we take advantage of an improvement in precision to reduce cost rather than increase precision. For example, if we develop a more precise photographic and plotter system, for a given scale photography we can produce a more accurate map than possible with our older system. On the other hand, with our new system we could obtain the same accuracy as before but with photography of a smaller scale and hence at a considerable economic saving. Therefore, many of the developments we previously mentioned with respect to higher precision are often reflected in a reduction in cost and time.

Manual Efforts

Although photogrammetry offers a considerable saving in cost and time over conventional civil engineering methods of surveying and mapping, there is considerable need for research and development in this area. One of the major advantages of photogrammetric methods over ground surveying method is the reduction in manual effort. However, on analyzing the photogrammetric method, we find manual steps playing a dominant role. The three principal manual steps in photogrammetry are (a) ground control surveys. (b) stereocompilation, and (c) drafting. In each case skilled technical manpower is required. Since we face an acute shortage of competent personnel in these areas and probably will for years ahead, reduction of the manual effort required in photogrammetry offers one of the most fruitful areas of research and development.

Ground Control

In reduction of manual ground control efforts, two significant trends are taking place. First, we are experiencing a period of intense interest in aerial photogrammetric triangulation or control extension, primarily for horizontal control but also for vertical control. Since precision control extension is the role of the first order plotter, whereas until recently these instruments have been rare in private practice, it is expected that the near future will show an expansion limited only by the manufacturers' delivery rates. In addition to the instrument approach to control extension, considerable research and development activity is underway in the area of analytical extension of control. This approach has been rendered feasible through the advent of the electronic computer and undoubtly will play a significant role in the future. It is reasonable to expect that third order horizontal control surveys, especially for mapping, will soon be the domain of photogrammetry.

The second trend in ground control surveys is the application of new technological developments in electronic and optical instrumentation. Firms engaged in photogrammetric mapping have been particularly progressive in this area, showing relatively little inertia to change. It is expected that the Geodimeter will very soon play an important role in establishing the primary ground control for photogrammetric mapping. Such electronic systems as Shoran and Raydist have already proven their value for major control networks. Through continuous research and development it is expected that the

perfection of electronic surveying systems will continue and will materially assist in the manual phase of ground control surveys.

Stereocompilation

The manual requirements of the stereocompilation phase of photogrammetric mapping is a real challenge to research. Although part of the function of the human stereoplotter operator is mechanical in nature, human judgment plays an important role. Applying automation to the complex judgment and interpretation role of the human element in stereocompilation offers problems of tremendous scope. In this area of research, electronics is beginning to play an important role in photogrammetry. Research has been underway for quite some time on a completely automatic stereoplotting system in which the human element is replaced by electronic systems. Such investigations, which have reached the prototype stage, are being conducted by Engineer Research and Development Laboratories and the Hycon organization. Another example of research activity in the direction of photogrammetric automation is the work of Paul Rosenberg Associates conducted for ERDL. Their efforts have been directed toward applying information theory to photogrammetry which has resulted in pointing the way to many new avenues which photogrammetry may take in the future. This is not the type of research which will change photogrammetry overnight. The difficulties which are exposed defy immediate solution. However, it is an extremely fine example of the breadth and depth of current photogrammetric research.

Drafting

The third area of major manual effort in convential photogrammetric mapping—drafting—perhaps does not excite the imagination of the research engineer but it is an area of considerable economic importance in terms of both cost and manpower consumption. The final drafting phase may contribute as much as 30% of the cost and time involved in a photogrammetric mapping project. In the case of many engineering maps, this final drafting is demanded to improve upon the neatness of the map compilation manuscript. One step in the direction of reducing the manual drafting effort would be for the engineer—user to eliminate his demand for "an ink tracing on linen." Our collegues in many design fields have ceased using ink tracings in favor of faster and more economical drafting methods. Certainly when a map is to be used only for a location study, the neater appearance of an ink tracing is hardly worth the additional time and cost. Many location studies could be conducted in terms of the map compilation manuscript, completely eliminating the final map drafting stage.

One advantage in the area of final drafting has been the development of plastic scribing as a substitute for ink tracing. Research and development on such cartographic method has been conducted by such organizations as the U.S. Geological Survey and the Aeronautical Chart and Information Center. Plastic scribing and related developments are primarily applicable to color separation drafting but have been applied to engineering map drafting in some cases. Scribing techniques speed up but do not replace manual requirements. There is ample room for research and development of automation systems to replace the human element in map drafting.

New Applications

Design Surveys and Maps

During the past decade, the use of photogrammetry in the planning and location phase of civil engineering projects has become almost universal. The new application emphasis is now on the design phase. There is a rather conclusive trend toward the use of photogrammetry for all necessary surveys and maps to completely design engineering construction projects such as highways. The ability to produce a complete set of detail highway construction plans including final quantities without convential ground surveys has already been demonstrated. In a relatively few years, the use of photogrammetry for design surveys and maps will also be universal.

Precision design mapping by photogrammetry has of course presented many new problems to the photogrammetric engineer. One of the more serious problems is obtaining suitable photography from the extremely low flight heights demanded by small contour intervals and elevation tolerances. Not too many years ago we considered 5,000 feet as low altitude for aerial photography. Today we are talking about flight heights of 1200 to 1800 feet. In this range, air turbulance, image motion, cycling time, and many other factors take on added importance. In addition, whereas in planning maps, vertical or contour error usually dictates the flight height, when we talk about design maps with a scale as large as 1" = 40' or even 1" = 20', horizontal tolerance begins to control flight height. Ground cover is an extremely critical limitation in design maps. These and many other factors offer considerable ground for extensive research and development in photogrammetry of importance to civil engineers.

Land Surveys

Another area of application of related interest to civil engineers is the application of photogrammetry to land surveys. This area has been developed to a rather high degree in Europe. Property corners are signalized before the photography is taken and spatial coordinates are obtained using a first order stereoplotter. Position location of corners with an error of only a few inches has been reported. Although the land use pattern and property survey problems are quite different in Europe, their work is an interesting example of the continuous improvement of photogrammetry. It is understood that the U.S. Forest Service is conducting some very interesting research on the use of photogrammetry for cadastral surveys in relocating corners. If this effort is successful it may be a major step toward use of photogrammetry for land surveys in the United States.

Laboratory Photogrammetry

New applications of photogrammetry to civil engineering are by no means restricted to surveying and mapping. Just as in many other scientific and engineering fields, photogrammetry is being used as a laboratory measurement method in many areas of civil engineering research including structures, hydraulics, soils, pavements, and similar subjects. Such applications of photogrammetry offer a fascinating challenge to the photogrammetric research engineer since the conditions and requirements are usually entirely different for each project. A very sound background in photogrammetric

fundamentals is necessary to design an optimum system to meet an entirely new set of conditions.

Photogrammetric Research and Highway Engineering

Impact of the Highway Program

The major impetus to research and development in photogrammetry as related to civil engineering will be the new expanded highway program. Photogrammetry will play a major role in accomplishing the engineering demands of the program. The highway program in turn will have a great impact on photogrammetry. Although most research and development in photogrammetry has previously been conducted by or sponsored by the military and national mapping agencies of the federal government, the magnitude and importance of the highway program justifies a sizeable research program in photogrammetry as directly related to highways, both at the national level and by the individual states.

Although an accurate estimate is impossible, estimates of the expenditures by highway departments for photogrammetric services during the next ten years indicate an order of magnitude of $200,000,000 although it could easily reach twice this amount. Due to the dynamic nature of photogrammetry and the tremendous potential for technological advancements, an expenditure the equivalent of five per cent of this amount for research and development in photogrammetry is entirely reasonable. Therefore, we should be thinking in terms of a collective research expenditure of $10,000,000 or $1,000,000 a year during the next decade. Although this sounds like a lot of money, it is only approximately one-thirtieth of one per cent of the federal highway expenditures alone. Whether or not the value of photogrammetric research to this degree will be recognized remains to be seen. However, we can expect to see some significant changes in civil engineering photogrammetry as a result of the highway program.

Role of the Engineering Schools

It is felt that cooperative research between the highway departments, photogrammetric firms, and the engineering schools should be developed. Participation by the engineering schools should be strongly encouraged. Sponsoring photogrammetric research in the engineering schools will serve the double purpose of (1) making effective utilization of the research experience and facilities of the schools and (2) encouraging the strengthening of the education programs in photogrammetry at the graduate level. It is felt that the latter purpose is of most importance. The future of photogrammetry and its potential contribution to civil and highway engineering depends almost entirely on the number, caliber, and educational background of the young engineers who choose to enter the field.

In recent years, a number of engineering schools have taken steps in the direction of strengthening their photogrammetric programs. This trend should be actively supported by the civil, highway, and photogrammetric engineering professions.

M.I.T.—Massachusetts Department of Public Works Program

As an example of one step in the direction of cooperative research suggested by the writer, a brief description will be given of the current research program at M.I.T. sponsored by the Massachusetts Department of Public Works. The mission of the program is broadly defined as ". . . research on the use of photogrammetry to conserve engineering manpower and reduce time and costs. . . ." Current activities encompass the following phases:

(1) Technical Information—Within the scope of this phase, the research staff is working with selected groups and individuals of the Department in making available to them technical information on the theory and application of photogrammetry to highway engineering. Group and individual conferences are held frequently.

(2) Current Practices—The research staff is available to the Department in a technical advisory capacity to review current practices and problems. Informal reports are prepared on problems and questions referred to the research staff by the Department.

(3) Photogrammetric Accuracy Factors—with the expanding utilization of photogrammetry for design maps and construction quantities, the need for more information on accuracy is increasingly apparent. Work is underway in this area to give the Department some first hand information on accuracy factors.

(4) Photogrammetric Cost Factors—Efficient planning of photogrammetric projects requires an understanding of the cost and time factors involved in executing a project. The time factors are particularly important. A guide to such cost and time factors is being prepared by the research staff for the assistance of the Department.

(5) Long Range Research—The activity of this phase is concentrated on the integration of photogrammetric systems and electronic computer systems for the development of semi-automatic data procurement—data reduction approaches to highway engineering. Due to the importance and scope of this subject, it will be discussed in the remainder of the paper.

Integration of Photogrammetry and Electronic Computers

Photogrammetry is basically concerned with the procurement of spatial data. Electronic computers are concerned with data processing. By integration of photogrammetry and electronic computers, we mean the development of a continuous data handling system in which each step and phase is a well coordinated link in the flow process from raw information to final answer. The ultimate goal is to eliminate or reduce the many manual data procurement and data reduction steps in convential highway engineering practice. In addition, more scientific and thorough approaches to highway planning, location, and design may be made possible by increased capacity to procure and analyze larger masses of data.

Evaluation of Highway Engineering Requirements

In the process of investigating new approaches and methods of procuring, storing, reducing, and analyzing data in the highway engineering field, the desirability of a complete re-evaluation of the basic data requirements for planning, designing, and constructing a modern highway has become increasingly apparent. The common question asked the photogrammetric engineer

is "What data can you furnish and what accuracy can be achieved?" It is felt the more basic question is "What data is really needed and what accuracy is actually required?" The problem is to clearly differentiate between essential information and information which can only be justified by the doctrine of tradition.

As an example, we may consider the matter of earthwork and cross sections. What information is basic and what information is traditional? In the matter of accuracy of earthwork quantities, what is a reasonable error tolerance? An interesting question is "What error level are we now tolerating in earthwork quantities obtained by convential ground surveys and cross-section area practices?" Such methods are of course approximations to the exact values using one of many possible ways of statistically representing a strip of topography.

One of the major problems the computer engineers have faced in applying modern computing equipment to highway practice is the tremendous variation in the fine detail of procedures and practices developed by each of the forty eight states over the decades. Although universal systems are too much to hope for, in the development of entirely new approaches and methods of highway data procurement and reduction, an excellent opportunity is available for modernizing the whole structure of highway engineering practice.

Analog and Digital Data

In integrating photogrammetric stereoplotters and electronic digital computers, we are dealing with two different types of data. The stereoplotter creates a model or analog of the terrain in which our data is characterized by continuous lines or surfaces. The electronic digital computer works only with digital or numerical data corresponding to discrete points of the analog. Therefore, one of our first requirements is to be able to readout digital data from the stereomodel. This facility is already available on several of the first order plotters. On double projection plotters such as the Kelsh and Balplex, we have a Z-readout but no XY-readout. Several possible methods of obtaining XYZ-readouts on a double projection plotter have been investigated by the M.I.T. research group with current activity centering on the Nistri electro-co-ordinatometer and companion units. This equipment has the advantage that it is already commercially available and can be applied directly to existing plotters without modification. The XYZ motions in the model are transferred to direct readout or printout units by servo motors. Although the equipment is of Italian design and manufacture, the servos are supplied by Bendix Aviation Company. Arrangements are currently being made for the installation of this equipment on the Kelsh Plotter at M.I.T. for test project work.

Input—Output Considerations

The goal of eliminating as many manual steps as possible means that it would be desirable for the data to come out of the photogrammetric system in a form ready for direct submital to the electronic computer. For our goal of a continuous data handling system with few or no intermediate manual steps, we would want our photogrammetric data output to be in the same form as our electronic computer data input. This would take the form of punched cards, punched tape, or magnetic tape, depending on the type of computer.

Despite the apparent advantage of punching out data directly at the time of

procurement, the basic consideration is overall economy. Since the data procurement phase operates at a much slower speed than the data processing phases, a continuous flow may not be the most economical system. For example, if one card punching machine can handle the output from half a dozen stereoplotters, printout with a manual punching step may be more economical than direct punchout.

Visual Presentation of Data

An additional factor in the plotter-computer integration effort is the need for visual inspection and control of the data flowing from the plotter to the computer. This can take the form of a graphical plot of the variables and/or a printout of digital data. In the case of plotters with coordinatographs, the data can be continuously plotted while readouts are being obtained for the computer. The same mission is accomplished with the Nistri equipment on double projection plotters by attaching a servo slave unit to the master unit. Another approach is to represent the variables on an oscilloscope screen which can be photographed.

In the area of visual presentation of data, there is considerable room for fresh approaches in highway engineering. Here again, the question of essential versus traditional is posed.

Terrain Representation and Storage

Although terrain is composed of infinitely complex continuous surfaces, we have noted that the electronic computer can only work with discrete points. Hence a statistical representation of the terrain is required in the photogrammetric-computer method just as in the case of convential ground methods of surveying topography. A number of choices are available for selecting the system of points to be used for statistical representation of the topography. These include (a) random points, (b) points on a square grid, (c) points on a rectangular grid, (d) random points on parallel lines, (e) random points on random lines. Many factors influence the choice of system and the density of points for a selected system. Some of the factors involved include (a) the degree of approximation permitted, (b) the nature of the terrain, (c) method of taking the data out of the stereomodel, (d) data storage system, (e) type and capacity of computer used. There is a need for considerable research in these areas. Our traditional topographic map is a highly efficient and simple system of representing the terrain. When we attempt to represent this same information in numerical form, we are faced with a major task.

It would be highly desirable to have a system which would require only one data procurement pass but be completely flexible within reasonable spatial limits with regard to the horizontal and vertical location of the highway. This is simple with respect to vertical flexibility, corresponding to a simple grade line change with convential cross-section data. However, horizontal flexibility presents a major problem. If we want to use the same set of terrain data for any number of widely separated and entirely different alignments, we are faced with a major data storage problem and spatial relationship problem. For this reason, a systematic distribution of the points statistically representing the terrain offers a major advantage over a random distribution.

Semi-Automatic Data Systems

In working toward the goal of a semi-automatic data procurement—data reduction system for highway engineering through the use of photogrammetry and electronic computers, the research staff of the M.I.T. Photogrammetry Laboratory has been conducting preliminary investigations of the many considerations outlined above. In many cases, the conception of an ultimate system presents technical problems which defy apparent solution based on existing knowledge and instrumentation. Hence, considerable research must proceed development.

Several other groups are working in the area of integrating photogrammetry and electronic computers. The Ohio Department of Highways has done considerable work in this field and will have an actual operational system in use in the intermediate future which will represent a major step in the direction discussed. The private engineering firm of Lockwood, Kessler, and Bartlett, Inc. is also taking a leading position in the integration program. It is expected that many other progressive state highway departments, private engineering firms, and equipment manufacturers will very soon recognize the technical and economical implication of this subject which will serve to expedite the goals of semi-automatic data systems. It is hoped that the civil engineering profession will take an active part in this program.

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PROGRESS REPORT OF THE COMMITTEE ON HIGHWAY AND BRIDGE SURVEYS OF THE SURVEYING AND MAPPING DIVISION:

Foreword to Manual on Highway and Bridge Surveys and Chapter I -- State Plane Coordinates

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FOREWORD

Introduction

The design and execution of highway surveys has failed to keep pace with the ever-increasing volume of new highway construction and with the more complex and critical construction design standards now in use. The sudden emergence of the nation's largest single roadbuilding program, made possible by the passage of the Federal Aid Highway Act of 1956, following a decade of intensified activity in toll road development and tax-supported systems of superhighways focusses new attention on the necessity for studying and redefining the fundamental surveying requirements of the modern highway project.

No longer is it economically defensible to execute highway surveys with the short-range objective of serving only the immediate needs of construction nor to base them on assumed horizontal and vertical datums. The modern highway survey when governed by high accuracy standards and when properly monumented and described can provide for all time adequate reference controls for future reconstruction and betterment programs. When tied into and properly adjusted to the national systems of horizontal and vertical control, with horizontal positions expressed in terms of state plane coordinates, such surveys can serve the country's domestic economy and aid her defense in the event of war. In the latter case state plane coordinates can be transformed into Universal Transverse Mercator coordinates for use by the Department of Defense.

With the advent of aerial photogrammetry and the growing shortage of highway engineering personnel there has developed the necessity of performing a substantial part of highway and other route surveying work with

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photogrammetric procedures. An associated development is the use of electronic computing methods to accelerate the determination of earthwork quantities in the study of alternate routes and to perform a variety of surveying calculations such as the computation and adjustment of traverses.

The need for more accurate field surveys, traverse and levels, to control the stereocompilation of topographic maps used in highway location studies and for coordinating the several subdivisions of extensive highway projects, not uncommonly 100 or more miles long, has led to the necessity of defining and disseminating new and higher specifications for such surveys. The planning, design, and layout of intricate interchanges and multiple-level grade separation projects has placed additional emphasis on the need for more accurate surveys.

In recent years the spectacular increase in the number and length of overwater highway crossings makes essential the description of related engineering procedures for executing surveys of large bridges. This topic and another on other special surveys such as those for vehicular tunnels are the subjects of separate chapters.

The principal purpose of this manual is to provide a working guide for the highway engineer in designing and executing adequate and lasting survey controls for highway and bridge construction. It attempts to fulfill this function by (1) explaining and demonstrating the advantages of state coordinate data as the logical basis for horizontal position referencing, (2) emphasizing the importance of using a standard reference surface, the 1929 Mean Sea Level Datum, for elevations, and (3) marshalling and explaining the classic steps of Reconnaissance, Preliminary Survey, and Location Survey, as modified by the latest advances in the art of surveying, as the only logical basis for the orderly and economical study, design, and location of new highways and highway bridges.

Elements of the Highway Survey

Highway surveys are executed to secure the essential data needed for the design of new construction or the improvement of existing facilities. Such surveys should be governed by standards of accuracy. The specifications for aerial as well as ground surveys are discussed in the body of this manual and summarized in the appendix.

Highway surveys should be carefully planned, designed, and executed in order to economically and adequately obtain the information necessary for optimum location, design, and construction of all classes of vehicular roadways. Although such surveys are not always executed in the three classic steps traditionally associated with the study and location of any route of transportation, it is considered most desirable to treat of these three phases, viz., reconnaissance, preliminary survey, and location survey.

The reconnaissance entails the preparation of a complete report embodying the essential information affecting a proposed undertaking. Reconnaissance surveys, both ground and aerial, are essential in securing sufficient data to permit the intelligent assessment of all features of the broad belt of country within which the proposed route is to be situated. Reconnaissance studies may be characterized as general rather than detailed studies of route location.

After review of the reconnaissance report and the selection of design standards for the new road, the preliminary survey phase is entered. Fundamentally, this consists of the systematic and detailed study of one or more

routes joining the specified termini of the project. Photogrammetric procedures are quite often employed in this phase of the work. The end product of the preliminary survey is frequently the projection on a strip topographic map of the centerline of the proposed highway and the preparation of preliminary cost estimates.

The location survey begins with the translation to the ground of the centerline location determined from the preliminary survey study. As the staking proceeds, some tolerance is provided in best fitting the line to the terrain. The survey continues with the execution of profile leveling and cross-sectioning in order to permit the laying of a final grade line. When photogrammetric procedures are utilized in the highest degree, end-areas and volumes of earthwork for specific gradelines are determined by use of stereoplotting machines and electronic computing methods. Other pertinent data that are secured in the course of the location survey are the legal descriptions of parcels of land that must be purchased for right-of-way, details of crossings with existing highways and railroads, location of existing utilities, information on drainage, and site data for bridges and culverts. Detailed surveys are made of interchange areas and of proposed borrow pit locations.

Chapter I

STATE PLANE COORDINATES

1. Introduction

Preliminary Remarks

The necessity of relating the horizontal positions of points in one survey to those of another by means of a common system of coordinates has assumed added importance in recent years. Modern civil engineering projects have increased greatly in complexity as well as areal extent and require more rigid geometric controls. This trend has underlined the need of converting existing geodetic positions to plane rectangular coordinates and of computing and expressing the results of precise horizontal control surveys in terms of some standard plane coordinate system. Nowhere has this need asserted itself with greater urgency than in the location phases of the programs of interregional highway construction that were begun in the early nineteen-fifties.

Before explaining the fundamental properties and use of state plane coordinate systems, it will be desirable to describe the national network of horizontal control, comment briefly on the historical development of state plane coordinate systems, and mention the major advantages of such systems as compared with a geodetic system.

National Network of Horizontal Control

The federal government has long recognized the necessity for an accurate network of horizontal control for the nation in order to perpetuate national, state, and other boundaries, provide a correlating framework for mapping and charting programs, and to serve many other purposes. The term, horizontal control, broadly refers to any system of survey points whose horizontal position coordinates have been determined. For the past 150 years the U. S. Coast and Geodetic Survey has engaged in operations leading to the establishment of the geodetic position of thousands of monumented points in all parts of the country. This governmental bureau is charged with the responsibility of establishing the basic first- and second-order horizontal (and vertical) control upon which most other control surveys in the nation depend. Fig. 1-1 tabulates the latest specifications for the three major classes of triangulation and traverse surveys. Various other federal agencies like the U. S. Geological Survey, Corps of Engineers, and the Forest Service have supplemented the fundamental net of first- and second-order control with extensive third-order control operations consisting of triangulation and traverse with the latter predominating.

Geodetic Coordinates

In the past it was the practice of the U. S. Coast and Geodetic Survey to publish and make available to interested engineers and surveyors the positions of triangulation and traverse points in terms of their spherical coordinates only, viz. geodetic latitude and longitude. Utilization of position data in this form required a working knowledge of geodetic engineering principles and computational processes unfamiliar to the local practitioner. The geodetically expressed positions, therefore, presented a formidable barrier to

acceptance of federal control data and inhibited any substantial use of such data or of the station monuments representing their physical embodiment.

It soon became apparent that a simple and practicable means of utilizing the highly accurate horizontal control data of the national network was needed. Prior to 1933 this problem was surmounted in the case of several large municipal topographic surveying projects, such as for New York City and Pittsburgh, Pennsylvania, by devising local systems of rectangular tangent-plane* coordinates. These systems served areas of limited extent satisfactorily but could not be extended over large regions without introducing errors of serious magnitude. This deficiency focussed attention on the mathematical development of statewide systems of rectangular plane coordinates.

State Plane Coordinate Systems

In order to make the position data of the national horizontal control system readily available in a more satisfactory form to local engineers and surveyors, the U. S. Coast and Geodetic Survey began in 1933 the establishment of rectangular plane coordinate systems for all states. These are commonly known as state coordinate systems.

Every state has its own coordinate system (Florida and New York have two systems) with each state covered by one or more belts or zones of the system. The basis of all the systems is the projection of a portion of the earth's surface upon the surface of a cone or cylinder which is subsequently developed into a plane. Horizontal position data now available from the Coast and Geodetic Survey include not only the geodetic coordinates but also the x and y rectangular plane coordinates (eastings and northings), in feet, on the appropriate state coordinate system. It is to be noted that just as there is but one point on the earth corresponding to a position expressed geodetically by latitude and longitude, there is likewise but one point which will satisfy a given pair of plane coordinates for a particular zone of a given state.

The substitution of the state coordinate systems and their associated position data for the relatively unwieldy goegraphic coordinates and unfamiliar geodetic computations has resulted in an increase in the use of federal control data and to a better appreciation of the advantages of referencing local and dependent surveys to the national network. Of great importance is the fact that the engineer who ties his carefully executed survey to first- or second-order triangulation or traverse may easily calculate and adjust his survey with the ordinary latitude and departure computations of plane surveying. While accomplishing this, he is simultaneously embodying in his survey the high accuracy and enduring qualities of the parent geodetic survey. It should be emphasized, however, that reliable results will be obtained only if the project survey is connected to first- or second-order control and is executed with instrumental equipment and procedures that will assure at least third-order accuracy.

There are some misconceptions about state plane coordinates that need to be dispelled. First, there is no special kind of control surveying that can be correctly termed state coordinate surveying. Any adequate operation of measuring distance and carrying azimuth from a point of unquestioned state coordinate position will make possible the determination of the position of other points. Second, there is no survey station that can be exclusively and uniquely

* ASCE Manual No. 10, Technical Procedure for City Surveys, offers an explanation of a tangent-plane coordinate system.

	First Order			Second Order		Third Order
	Class I	Class II	Class III	Class I	Class II	
PRINCIPAL USE	Urban surveys, scientific studies	Basic network of United States	State, county, private	Area networks and supplemental cross arcs in national net	Coastal areas and inland waterways. Supplemental and cadastral	Topographic
BASE MEASUREMENT						
Probable error not to exceed	1 part in 1,000,000	1 part in 1,000,000	1 part in 1,000,000	1 part in 1,000,000	1 part in 500,000	1 part in 250,000
TRIANGLE CLOSURE						
Average not to exceed	1"	1"	1"	1.5"	3"	5"
CLOSURE IN LENGTH						
Discrepancy between computed length and measured length of base, or adjusted length of check line, not to exceed.	1 part in 100,000	1 part in 50,000	1 part in 25,000	1 part in 20,000	1 part in 10,000	1 part in 5,000

(a) Triangulation

Fig. 1-1 Specifications for Triangulation and Traverse

	First Order	Second Order	Third Order
Number of azimuth courses between azimuth checks not to exceed . . .	15	25	50
Astronomical azimuths: Probable error of result	0.5 sec.	2.0 sec.	5.0 sec.
Azimuth closure at azimuth check points not to exceed.	2 sec. $\sqrt{N}$ or 1.0 sec. per station	10 sec. $\sqrt{N}$ or 3.0 sec. per station	30 sec. $\sqrt{N}$ or 8.0 sec. per station
Distance measurements accurate within	1 in 35,000	1 in 15,000	1 in 7,500
After azimuth adjustment, closing error in position not to exceed. .	0.33 ft. $\sqrt{M}$ or 1 in 25,000	1.67 ft. $\sqrt{M}$ or 1 in 10,000	3.34 ft. $\sqrt{M}$ or 1 in 5,000

N is the number of stations for carrying azimuth
M is the distance in miles

(b) Traverse

Fig. 1-1 Specifications for Triangulation and Traverse

described as a state corrdinate station. Defining the position of a point in any scheme of horizontal control by state coordinates is merely utilizing a mode of expressing location which is analogous to that of using geodetic coordinates. Corresponding to every pair of state coordinate values is a pair of latitude and longitude values and they are mutually interconvertible. Third, the quality of the state coordinate position of any monumented survey point is not inherently high merely because its northing and easting is related to a particular state grid system. State coordinate positions can be no better than the data from which they were derived. For example, x and y values derived from erroneous longitudes and latitudes or from a faulty traverse will be weak. Fourth, no horizontal control survey regardless of whether its results are expressed in terms of geodetic or state plane coordinates can have intrinsically cadastral or proprietary qualities. There are certain definite procedures known to the qualified land surveyor which must be used in the legal restoration of lost section and other boundary corners. However, the accurately known state plane coordinates of a lost corner may be the best collateral evidence of the position of the corner and offer the most efficient means for its restoration.

Advantages of State Coordinate Systems

Later in this manual specific applications of state coordinate systems to highway and bridge surveys will be described in order to make more evident their advantages. Here it will suffice to mention by way of introduction four fundamental benefits to be derived from the use of state coordinates. These benefits are inherited from the basic geodetic network upon which all state coordinate systems ultimately depend.

a) Positive checks can be applied to all dependent and local surveys to control errors in angles, distances, and positions.

b) Surveys for highways, canals, pipelines, power lines, etc., can be initiated at widely separated points with assurance that the separate sections of the project will fit together when all the component surveys are completed.

c) The state coordinate systems provide a good map base and an excellent means for indexing field notes, property descriptions, and other similar information.

d) Any survey station whose state coordinates have been once accurately determined may be said to be permanently located. Should its marker be destroyed, it is possible to restore the station by careful measurements from the nearest recovered survey point in the system.

2. State Grid Systems

General Considerations

State coordinate systems are in principle rectangular coordinate systems defined by a pair of perpendicular lines, called axes, situated in a plane. The perpendicular lines are the x axis and the y axis and their point of intersection is the origin. It is customary to position the x axis in an east-west direction and the y axis in a north-south direction. The position of any point in the plane is designated by its coordinates which represent the distances from the

axes to the point. These coordinates are usually termed x or the easting and y or the northing. The x coordinate of a point is the distance from the y axis to the point. The y coordinate of a point is the distance from the x axis to the point. The principles of plane analytical geometry can be freely applied to problems of state grid systems involving length and direction of lines, intersection of lines, and transformation by rotation.

Plane rectangular systems for surveying are obviously not new developments. Local systems of limited extent have been used by engineers and surveyors since ancient times. However, the correlation of isolated and arbitrarily defined systems on the earth's spheroidal surface presents some difficulties particularly when the enterprise such as a large highway construction project, extends over considerable distances. These problems relate chiefly to incompatibilities in orientation, scale, and grid coordinate values between two connecting systems. The state coordinate systems were created to overcome these difficulties.

Map Projections

Mathematical cartography cannot be considered here but it will be helpful to preface the following general description of state coordinate systems with some introductory remarks.

A map is a relatively small scale representation on a plane surface of some part of the earth's surface. Obviously the curved surface of the earth cannot be developed into a plane without some stretching or shrinking. However, the distortion can be controlled if the points on the earth's surface are projected upon a developable surface such as that of a cone or cylinder. Following such projection and development onto a plane surface, the points on the map will represent the nearly correct relative positions of the corresponding ground points.

Various projections suitable for rectangular plane coordinate systems have been devised but the adaptation of only two to surveying practice will be mentioned here. Both are classified as conformal projections because the shape of any small area of the surface being portrayed is preserved unchanged. Expressed in another way, the scale of a conformal map projection at any point is the same in all directions.

Lambert Projection

This is a conformal map projection of the so-called conical type on which all geographic meridians are represented by straight lines which meet in a common point outside the limits of the map and the geographic parallels are represented by a series of arcs of circles having this common point for a center.

The elemental characteristics of this projection which has been widely adopted as a basis for state plane coordinate systems are portrayed in Fig. 1-2. A secant cone whose axis is assumed coincident with the polar axis of the earth pierces the latter's sea level surface along two small circles, BB' and CC' which are termed the standard parallels and are situated approximately one-sixth the width (from north to south) of the zone from its northern and southern boundaries, DD' and AA', respectively. When the frustrum, ADD'A', is developed into a plane, it should be noted that the scale of the projection along the standard parallels is true, between them it is too small, and beyond them it is too great.

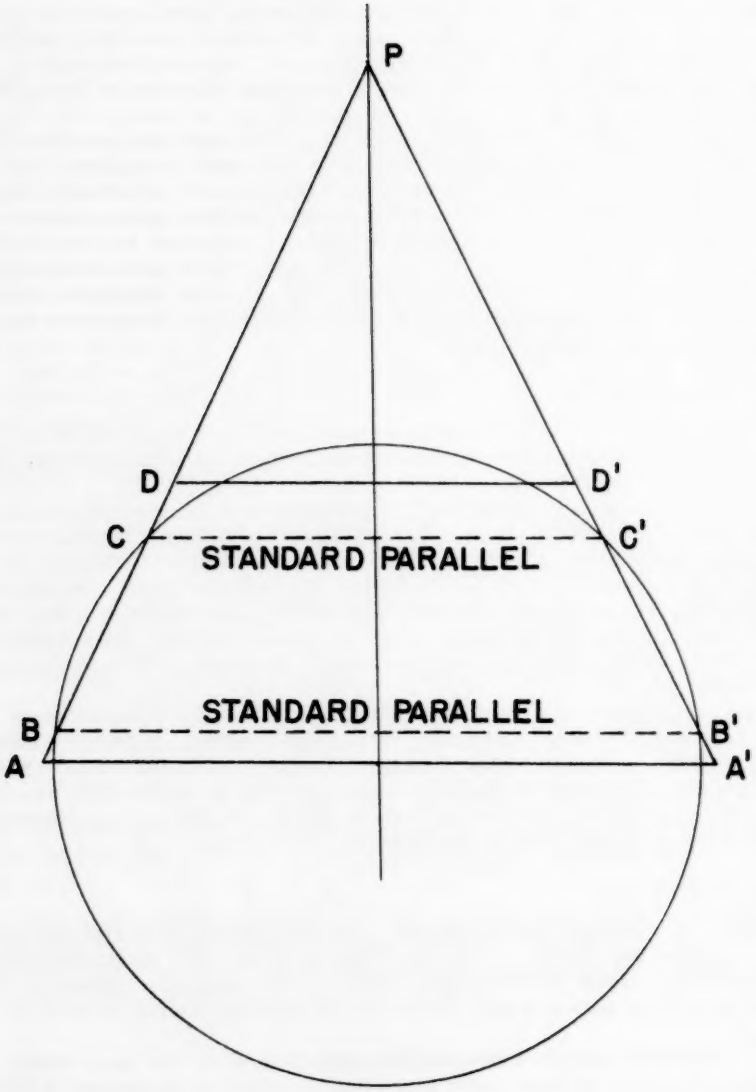

FIG. 1-2. LAMBERT PROJECTION

Transverse Mercator Projection

The Transverse Mercator projection is closely related to the Mercator projection. The latter is briefly described as the developed surface of a cylinder whose axis is coincident with the earth's polar axis and which has its surface tangent to the earth's mean sea level surface along the equator.

The Transverse Mercator projection is in principle similar to the Mercator projection which has been turned (or traversed) 90° in azimuth. Furthermore, the radius of the cylinder is slightly reduced so that instead of being tangent to the spheriodal surface along a central meridian, it cuts the earth's surface along two flanking small circles. Fig. 1-3 portrays diagrammatically an east-west section through the earth's surface and the plane surface of the Transverse Mercator System. It will be noted that the scale of the projection is true along the two flanking small circles, that it is too small for the area between them, and too large for the areas beyond them and up to the east and west limits of the projection.

Lambert and Transverse Mercator Grid Systems

State coordinate systems based on the Lambert and the Transverse Mercator projections are commonly designated as Lambert and Transverse Mercator grids, respectively. The former is generally used for those states whose greatest dimension lies in an east-west direction and the latter is usually employed for those states whose greatest dimension is in a north-south direction. Both projections are used in New York and Florida. Almost all states are divided into several belts or zones with each zone having its own origin and central meridian.

3. Transformation of Geodetic to State Plane Coordinates

Although it is now the practice of the U. S. Coast and Geodetic Survey to compute and publish the plane coordinates on the appropriate state grid system of all its triangulation and traverse stations, it will be advantageous to demonstrate the general nature of the machine computation for transforming geodetic to plane coordinates. This will be done for a Lambert System only in view of the fact that the government publications that contain the tables needed for making the transformation calculation also provide typical sample computations for both kinds of grid systems.

Lambert System (Wisconsin)

The transformation computation for a Lambert grid will be performed with reference to the Wisconsin State Coordinate System. As shown in Fig. 1-4 this state is covered by three overlapping zones, the North, Central, and South Zones. Each zone has different axes for x and y although all y axes, passing through the approximate center of the respective zone, are given an x value of 2,000,000 ft. The x axis is placed well below the southern limit of each belt and has a value of zero feet. The geodetic coordinates of triangulation station DENMARK in the south zone will be transformed into plane coordinates.

The South zone has for a central parallel the 43° 24' parallel of latitude. Along this line the scale ratio (error of projection) is 1 part in 14,800 parts too small. The standard parallels along which the scale is true are 42° 44'

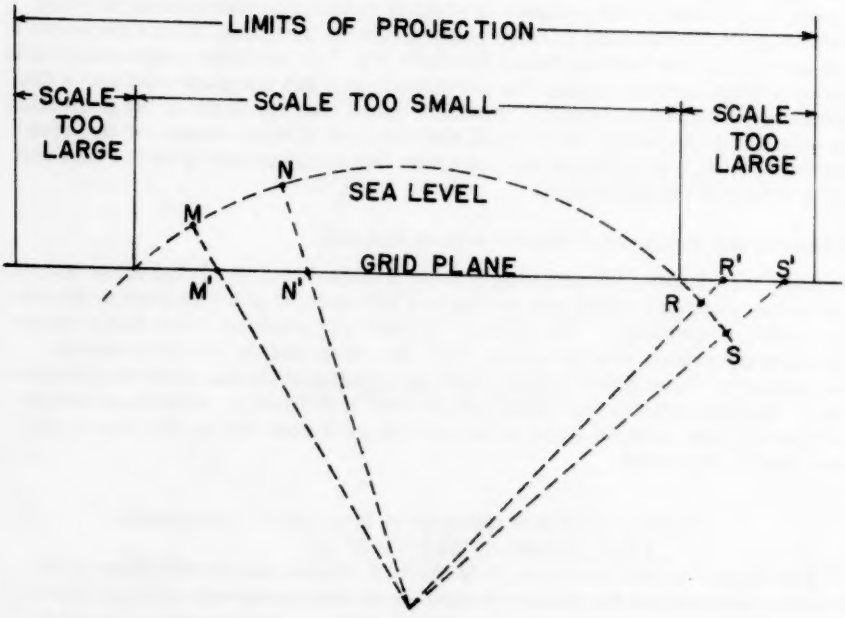
LIMITS OF PROJECTION
SCALE TOO LARGE
SCALE TOO SMALL
SCALE TOO LARGE
N
SEA LEVEL
M
GRID PLANE
R'
S'
M'
N'
R
S

FIG 1-3

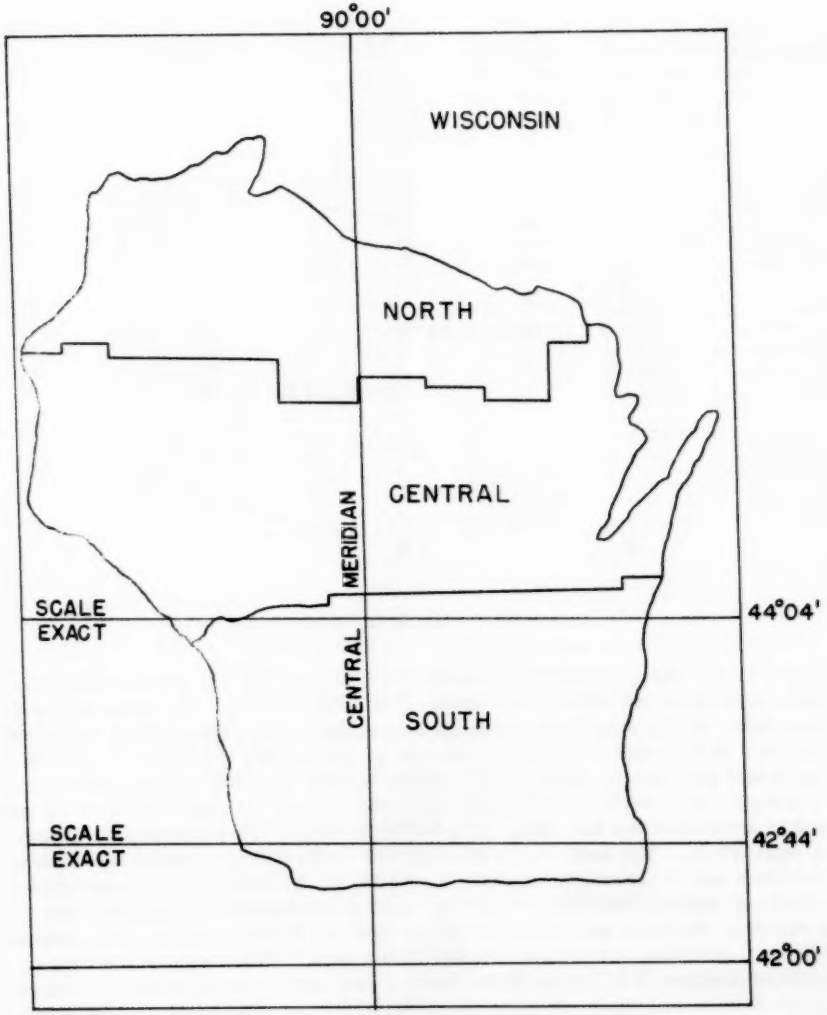
90°00'
WISCONSIN
NORTH
CENTRAL
MERIDIAN
SCALE
EXACT
44°04'
CENTRAL
SOUTH
SCALE
EXACT
42°44'
42°00'

FIG. 1-4

and 44° 04'. The meridian of longitude, 90° 00', is the y axis and its intersection with the 42° 00' parallel of latitude defines the point whose x and y coordinates are, therefore, 2,000,000 ft. and zero feet, respectively.

Fig. 1-5 portrays the basis for converting the geodetic coordinates of a

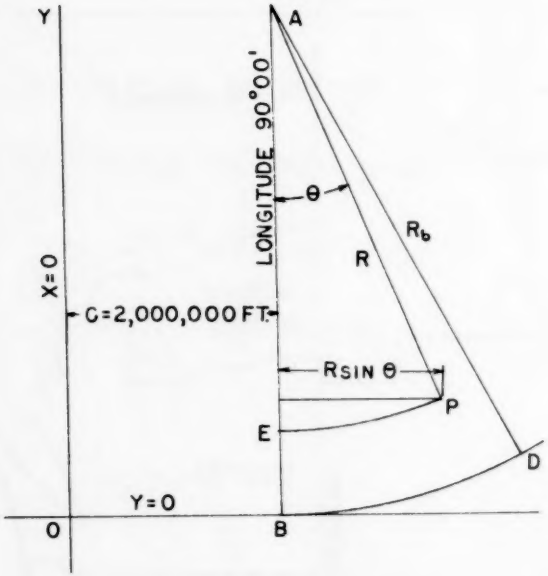

FIG. 1-5

point, P, into its Lambert grid coordinates. Point 0 is the origin of coordinates and AB is the central meridian of the grid system. The value of the x-coordinate of the central meridian is designated as C. The point A represents the apex of the cone on which the area is projected and the arcs PE and DB represent portions of parallels of latitude through point P and the lower extremity of the y axis. The distance R_b is the largest latitude radius of the zone and is a constant for the zone. It is to be noted that the y-coordinate of point A is equal to R_b. The angle θ, is the angle of convergency between the central meridian and the meridian through the point, P. Values of R for each whole minute of latitude, and values of θ for each whole minute of longitude are given in U. S. Coast and Geodetic Survey Special Publication No. 288, entitled "Plane Coordinate Projection Tables—Wisconsin." The angle, θ, is considered positive if P is east of the central meridian and negative if P is west of the central meridian. It is to be emphasized that grid north and geodetic north are identical along the central meridian.

Transformation Computation (Lambert)

By inspection of Fig. 1-5 the following equations can be written for point P:

$$x = R \sin \theta + C$$
$$y = R_b - R \cos \theta$$

A typical transformation computation will be performed using the pertinent portions of Special Publication No. 288 which are reproduced as Figs. 1-6 and 1-7.

LAMBERT PROJECTION FOR WISCONSIN—SOUTH ZONE

Lat.	R (feet)	y' y value on central meridian (feet)	Tabular difference for 1 sec. of lat. (feet)	Scale in units of 7th place of logs	Scale expressed as a ratio
44° 11'	21,876,351.30	795,783.36	101.26650	+112.0	1.0000258
12	21,870,275.31	801,859.35	101.26733	+129.5	1.0000298
13	21,864,199.27	807,935.39	101.26800	+147.3	1.0000339
14	21,858,123.19	814,011.47	101.26867	+165.5	1.0000381
15	21,852,047.07	820,087.59	101.26950	+184.1	1.0000424
44° 16'	21,845,970.90	826,163.76	101.27017	+203.1	1.0000468
17	21,839,894.69	832,239.97	101.27100	+222.5	1.0000512
18	21,833,818.43	838,316.23	101.27183	+242.2	1.0000558
19	21,827,742.12	844,392.54	101.27250	+262.3	1.0000604
20	21,821,665.77	850,468.89	101.27333	+282.7	1.0000651
44° 21'	21,815,589.37	856,545.29	101.27400	+303.5	1.0000699
22	21,809,512.93	862,621.73	101.27483	+324.7	1.0000748
23	21,803,436.44	868,698.22	101.27567	+346.3	1.0000797
24	21,797,359.90	874,774.76	101.27650	+368.2	1.0000848
25	21,791,283.31	880,851.35	101.27733	+390.6	1.0000899

Fig. 1-6

Given: Triangulation station DENMARK
Latitude 44° 20' 00.855", Longitude 87° 48' 04.616"
State -- Wisconsin; Zone -- South
C = 2,000,000 ft. R_b = 22,672,134.66 ft.

Required: Lambert coordinates, x and y.

Solution: R = 21,821,579.18 (Fig. 1-6)
θ = +1° 30' 38.6860" (Fig. 1-7)
$\sin \theta$ = +0.0263644386
$\cos \theta$ = +0.9996523977
then $x = R \sin \theta + C$
$x = (21,821,579.18)(+0.0263644386) + C$
$x = 2,575,313.71$
and $y = R_b - R \cos \theta$
$y = 22,672,134.66 - (21,821,579.18)(+0.9996523977)$
$y = 858,140.66$

In the foregoing calculation careful attention always must be given to the sign of the trigonometric functions. It is obvious a 10-bank computing machine and an expanded table of natural sines and cosines are required. U. S. Coast and Geodetic Survey Special Publication No. 246 contains tables of these functions for each second of arc to 10 decimal places. This publication and plane coordinate projection tables for all states are available at nominal cost from the Superintendent of Public Documents, Government Printing Office, Washington, D. C.

LAMBERT PROJECTION FOR WISCONSIN—SOUTH ZONE

1" of longitude = 0.68710324" of θ

Long.		θ		
87°	36'	+1°	38'	56".5720
	37	+1	38	15.3458
	38	+1	37	34.1196
	39	+1	36	52.8934
	40	+1	36	11.6672
87°	41'	+1	35	30.4410
	42	+1	34	49.2148
	43	+1	34	07.9887
	44	+1	33	26.7625
	45	+1	32	45.5363
87°	46'	+1	32	04.3101
	47	+1	31	23.0839
	48	+1	30	41.8577
	49	+1	30	00.6315
	50	+1	29	19.4053

Fig. 1-7

In much the same manner that geodetic coordinates are transformed to state coordinates, the latter can be converted to geodetic coordinates. Examples of the inverse computation are adequately presented in the government projection table publications.

Transverse Mercator System (Indiana)

The transformation from geodetic to Transverse Mercator coordinates is more difficult to explain. Since the detailed mathematical development of the formulas involved is not pertinent to this manual, it will not be attempted. If the engineer is faced with the need for performing the transformation computation, he should have no difficulty following the mechanics of the sample calculation which is illustrated in the publications containing the necessary tables. For the State of Indiana this is U. S. Coast and Geodetic Survey Special Publication No. 259 entitled "Plane Coordinate Projection Tables—Indiana." A brief description of the Indiana grid system seems desirable.

The Transverse Mercator Projection is sometimes described by stating that the axis of the tangent cylinder lies in the plane of the earth's equator. This projection can be illustrated by a cylinder cutting the surface of the spheroid along two ellipses which are equidistant from the central meridian of the zone. The cylinder is then considered to be cut along an element and developed into a plane.

Fig. 1-8 which has been reproduced from Special Publication No. 259 shows that the state of Indiana is covered by two overlapping zones, the East Zone and the West Zone. Each zone has its own y axis although both axes, passing through the approximate centers of the respective zones, are given an x value of 500,000 ft. Both zones use the same x axis which is situated well below the southern boundary of the state and has a y value of zero feet.

The central meridian of the West Zone is 87° 05' west longitude. Along this line the scale of the projection is 1 part in 30,000 parts too small (see Fig. 1-3). The lines of exact scale are parallel to the central meridian and situated approximately 32 miles (170,800 ft.) east and west of it. To the east and west of these lines respectively, the scale of the West Zone projection is too large. The intersection of the central meridian with the 37° 30' parallel of latitude defines the point whose x and y coordinates are 500,000 ft. and zero feet, respectively.

North American 1927 Datum

If the local engineer or surveyor performs the transformation computation for converting geodetic coordinates to state coordinates, it is highly important that geodetic position data based on the official national horizontal datum be used. This is the North American datum of 1927. It is possible the engineer may find in public and private libraries old publications containing position data referring to old astronomic and superseded geodetic datums. Also, he may encounter unadjusted or preliminary geodetic coordinates. The use of such data will lead obviously to incorrect state coordinate values.

Information concerning the availability, description, and state grid coordinates of horizontal control throughout the nation may be obtained on request from the U. S. Coast and Geodetic Survey, Washington, D. C.

4. Grid Azimuth

The projection lines of a state plane coordinate system, whether Lambert or Transverse Mercator, comprise what is termed a grid because all north-south lines are parallel with the central meridian and perpendicular to all east-west lines. Because of the convergency of the geographic meridians, it is apparent that the grid azimuth of a line will be the same as the geodetic azimuth only when the station at which the azimuth is expressed is located on the central meridian. For all other lines the grid azimuth will differ from the geodetic azimuth. This difference becomes greater with increasing distance of the survey point from the central meridian and is substantially equal to the angular convergency between the central meridian and the geographic meridian passing through the station.

Fig. 1-9 which has been prepared with reference to the central meridian of the Lambert South Zone for the state of Wisconsin indicates that for stations west of the central meridian, the grid azimuth is greater than the geodetic azimuth, and for stations east of the central meridian, the grid azimuth

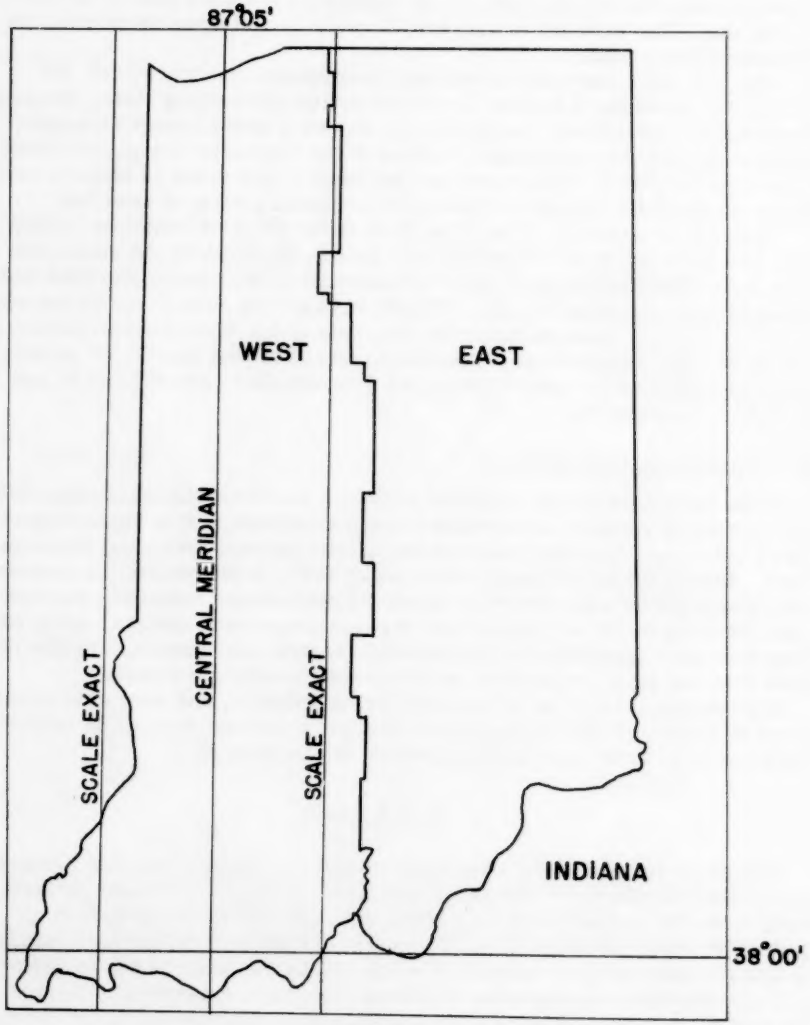
87°05'
WEST
EAST
CENTRAL MERIDIAN
SCALE EXACT
SCALE EXACT
INDIANA
38°00'

FIG. 1-8

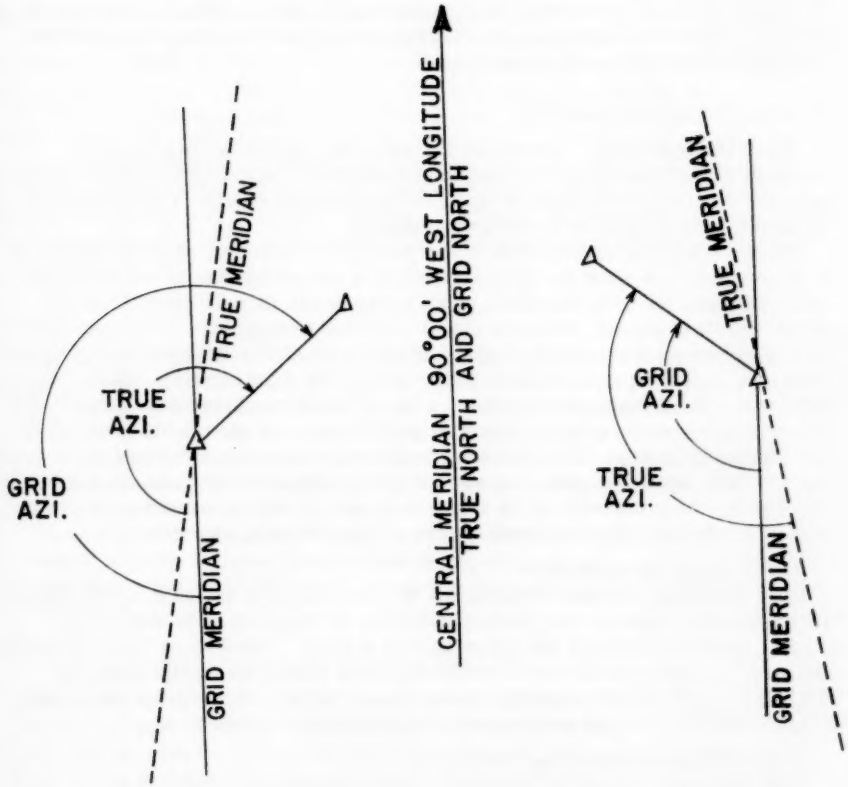
TRUE MERIDIAN
TRUE AZI.
GRID AZI.
GRID MERIDIAN
CENTRAL MERIDIAN 90°00' WEST LONGITUDE
TRUE NORTH AND GRID NORTH
TRUE MERIDIAN
GRID AZI.
TRUE AZI.
GRID MERIDIAN

FIG 1-9

is less that the geodetic azimuth. A clear understanding of the difference between grid (or plane) azimuths and geodetic (commonly called true) azimuths is of fundamental importance in engineering applications of state coordinates.

In a state coordinate system the forward and back grid azimuths of any line differ by exactly 180°. Such azimuths when appearing in government publications are most frequently reckoned from the south. The engineer must make certain he interprets correctly the given grid azimuths, i.e., whether reckoned from grid north or grid south.

Computation of Grid Azimuth

When the state plane coordinates of two stations are known, the grid azimuth of the connecting line can be determined from the principle that the tangent of the azimuth angle is equal to the difference in the x-coordinates divided by the difference in the y-coordinates.

Before the plane coordinates of the points in a traverse can be computed, it is necessary to know the grid azimuth of a reference line at the beginning station so that the grid azimuths of all courses can be calculated from the measured field angles. In some cases a distant triangulation or traverse station of known position can be sighted and the traverse initially oriented by the computed or known grid azimuth of the line to the distant point. Under other circumstances a sight can be taken on the azimuth mark which is a triangulation station accessory usually located at a distance of one-fourth to one-half mile from the station. It is common practice for the surveying agency to publish or make available upon request the grid azimuth to the azimuth mark. If only the geodetic azimuth to the mark is known, it will be necessary to convert this value to a grid azimuth by one of the following expressions:

a) For Lambert Systems—

Grid Azimuth = Geodetic Azimuth - θ + second term where θ is the angle of convergency between the central meridian of the projection and the geographic meridian through the given survey station. The second term is small and may be neglected for third-order traverse situations except when the orientation sight is considerably longer than 5 miles. The evaluation of this term is explained in the government projection table publication.

b) For transverse Mercator Systems—

Grid Azimuth = Geodetic Azimuth - $\Delta\alpha$ - second term where $\Delta\alpha'' = \Delta\lambda'' \sin\phi + g$

The quantity, $\Delta\lambda''$, is the longitude difference, in seconds of arc, between the central meridian and the survey point; ϕ is the latitude of the point, and g is a quantity obtainable from the appropriate projection tables. The second term is negligible for most situations.

Astronomic Azimuth

Occasionally neither geodetic nor grid azimuth will be available to the engineer beginning a control traverse for an engineering project. These circumstances make it necessary to determine the astronomic azimuth of the first course of the traverse by a solar or stellar observation and convert the resulting astronomic azimuth to a grid azimuth by one of the foregoing expressions. In doing this it is assumed the astronomic azimuth is equal to the geodetic azimuth. Only for the more precise geodetic surveys need the distinction between the two be maintained.

The determination of the astronomic azimuth of a line consists of two principal operations. The first is the field measurement of the horizontal angle between the star and a terrestrial signal, and the second is the office computation of the azimuth of the star at the moment it was observed. The combination of the two preceding angular quantities in an appropriate manner provides the astronomic direction of the line.

With the recent introduction of optical reading transits which are internally wired for night illumination of circles and cross-wires, and have horizontal circles reading to the nearest 0.1 minute of arc, the field work of observing Polaris is much simplified. Furthermore, the ease with which correct time can be obtained by radio, and the availability of star ephermerides permitting a streamlined calculation of the azimuth of the star make the entire problem quite routine. For an adequate explanation of the observing and reduction procedures reference is made to any of the standard surveying textbooks and the abbreviated ephermerides of several instrument manufacturers.

5. Grid Distance

Before proceeding with the calculation of an illustrative traverse problem, it will be essential to obtain an understanding of the term, grid distance, and of the distinction between grid distance and ground distance. In order to compute the grid coordinates of the stations in a traverse, it is necessary (1) to reduce all ground distances to mean sea level to secure their equivalent geodetic lengths, and (2) to transform such geodetic distances to grid distances on the plane of the state projection system.

To begin, it is necessary to apply to the observed field distances any corrections, such as for temperature and slope, as may be necessary. The resulting corrected field distances, termed ground distances, are then reduced to sea level.

Converting Ground Distance to Geodetic Distance

Reduction to sea level is facilitated by the tables in Fig. 1-10 which list in part (a) the factors by which the ground distance at various elevations is to be multiplied in order to obtain the geodetic distance, and in part (b) the corrections to be subtracted from a ground distance of 1000 ft. at different elevations in order to obtain the sea level or geodetic distance. Approximate elevations are satisfactory for calculating this correction. To give quantitative significance to the sea level correction it is to be noted that an error of 500 ft. in the elevation of a traverse course will cause a proportional error in its sea level length of only 1 part in 41,800 parts. Lack of consistency obviously characterizes meticulous office calculations of geodetic distances from field distances when the latter are inherently weak because of faulty taping or neglect of the temperature and other corrections.

Converting Geodetic Distance to Grid Distance

The term, scale factor, is defined as the ratio of the plane or grid distance of any line in a state plane coordinate system to the geodetic or sea level length of the line. The scale factor embodies the effect of distortion or linear change in a line on the sea level surface of the earth when the termini of the line are projected onto the plane of the projection. The scale factor may be

Elevation (Feet)	Elevation Factor	Elevation (Feet)	Elevation Factor
Sea level	1.0000000	3000	0.9998565
500	0.9999761	3500	0.9998326
1000	0.9999522	4000	0.9998087
1500	0.9999283	4500	0.9997848
2000	0.9999043	5000	0.9997609
2500	0.9998804	5500	0.9997370

(a) Elevation Factors

Elevation (Feet)	Correction Factor	Elevation (Feet)	Correction Faction
Sea level	0.0000	3000	0.1435
500	0.0239	3500	0.1674
1000	0.0478	4000	0.1913
1500	0.0717	4500	0.2152
2000	0.0957	5000	0.2391
2500	0.1196	5500	0.2630

(b) Elevation Corrections, in Feet per 1000 Ft.

Fig. 1-10 Reduction to Sea Level

TRANSVERSE MERCATOR PROJECTION
INDIANA
Both Zones

x' (feet)	Scale in units of 7th place of logs	Scale expressed as a ratio
0	-144.8	0.9999667
5,000	-144.7	0.9999667
10,000	-144.3	0.9999668
15,000	-143.7	0.9999669
20,000	-142.8	0.9999671
25,000	-141.7	0.9999674
30,000	-140.3	0.9999677
35,000	-138.7	0.9999681
40,000	-136.9	0.9999685
45,000	-134.8	0.9999690
50,000	-132.4	0.9999695
55,000	-129.8	0.9999701
60,000	-126.9	0.9999708
65,000	-123.8	0.9999715
70,000	-120.5	0.9999723

Fig. 1-11

readily found by referring to the appropriate part of the U. S. Coast and Geodetic Survey projection tables for the particular state. For all Lambert Systems the table of scale factors (see Fig. 1-6) is entered with the latitude. For all Transverse Mercator Systems the table of scale factors (see Fig. 1-11) is entered with the quantity, x', which is the distance from the central meridian.

The scale factors are expressed both as a ratio and as the correction, in units of the 7th place of logarithms, to the sea level length of the line. The ratio and the sign of the logarithm are those to be used for converting a geodetic distance to a grid distance. If the conversion is to be made in the opposite direction, the sign of the logarithm should be reversed and the reciprocal of the published scale factor should be employed.

When the scale factor is less that unity, and a calculating machine is not available, it is more convenient to subtract the tabular value of the scale factor from 1, and apply a subtractive correction to the geodetic length. For example, if the scale factor is 0.9999684 and the geodetic length is 2,841,85 ft., the grid distance is equal to

$$(2841.85) - (2841.85)\,(0.0000316) \text{ or } 2841.85 - 0.09 = 2841.76 \text{ ft.}$$

In order to determine the scale factors it will be helpful to make a plot of the traverse to some convenient scale such as one inch equals 4000 ft. The traverse can be plotted by means of the known grid coordinates of the first station and the ground lengths and preliminary azimuths of the various courses.

For Lambert Systems it is necessary to determine the average latitude of each traverse course. This can be satisfactorily done by scaling the distance, measured parallel to grid north, from the first station, whose latitude is known, to the middle of each course. This distance in feet can be converted with sufficient accuracy to seconds of latitude by dividing by 100. The resulting latitude difference, in seconds of arc, can then be applied to the latitude of the initial traverse station in order to determine a latitude for entering the projection tables (see Fig. 1-6) and ascertaining the scale factor by interpolation. In many cases it will be satisfactory to obtain an average latitude from the best available map and use it for the entire traverse.

For Transverse Mercator Systems a similar plot can be drawn to determine the mean value of x' for each traverse course. It should be recalled that if the x-coordinate of the beginning traverse point is 485,000 ft. in the Indiana system-West zone, the value of x' is -15,000 ft. The scaling of east-west distances from this point to the middle of each course will permit the determination of satisfactory values of x' for entering the appropriate part of the projection tables (see Fig. 1-11). All recently published topographic maps of the U. S. Geological Survey will be found to provide a convenient base for plotting traverses to determine scale factors because they contain marginal ticks for drawing the grid lines of the state coordinate system.

Grid Factor

It will be observed that the elevation factors of Fig. 1-10a and the scale factors of Figs. 1-6 and 1-11 are quantities by which the ground distance and geodetic distance, respectively, are multiplied in order to secure the grid distance. Since the engineer is not particularly interested in the geodetic distance, it is most efficient to use the product of these two factors for directly

computing the grid distance. This combined factor is termed the grid factor. For most work a mean grid factor can be used over a fairly large area such as a small county.

6. Traverse Computation

In order to clarify the use of the principles explained in the preceding articles, the detailed computation of a second-order, transit-tape traverse will be explained. Such a traverse may be considered to provide the horizontal control for a comprehensive highway mapping and construction project or, generally, for any extensive route engineering enterprise. After giving the essential data for the traverse, the usual steps in its computation and adjustment will be explained.

Control Data

The traverse depicted in Fig. 1-12 was executed between two fixed points of second-order horizontal control in east-central Wisconsin. The measured angles-to-right and ground distances are tabulated in Figs. 1-13 and 1-14. The elevations of the traverse courses, as determined from the Denmark, Wisconsin quadrangel topographic map of the U. S. Geological Survey varied over a range of only 170 ft. An average value of 760 ft. was used in the determination of the elevation factor.

The traverse was begun at triangulation station GIBSON for which the following description and survey data are given:

Station is located 8.0 miles southeast of Denmark (Manitowoc County) and 1.85 miles southeast of the junction of U. S. Highway 141 and State Highway 147, on the west side of U. S. Highway 141. It is on the highway right-of-way, 31 feet west of the center of Highway 141, 5 feet northeast of a white witness post, and 2 feet east of a fence line. Mark is standard U. S. Coast and Geodetic Survey triangulation disk stamped GIBSON 1953 and is flush with ground surface.

Reference Mark No. 1 is located 32 feet west of the center of Highway 141 and 1 foot east of a fence line. Mark is flush and disk is stamped GIBSON No. 1, 1953 (distance from triangulation station to reference mark is 32.82 ft.)

Reference Mark No. 2 is located 32 feet west of the center of Highway 141, 4 feet north of a 4" x 4" concrete post, and 1 foot east of a fence line. Mark is flush and disk is stamped GIBSON No. 2, 1953 (distance from triangulation station to reference mark is 28.18 ft.)

Azimuth Mark is located 30 feet east of the center of Highway 141, 3 feet south of a white witness post, and 1 foot west of a fence line. Mark is standard azimuth disk stamped GIBSON 1953 and is flush with ground surface.

Plane coordinates (Wisconsin-south Zone):

x = 2,586,009.52 ft.
y = 828,714.15 ft.

Grid azimuth (from south) to Azimuth mark = 337° 59′ 24"

The traverse extended for approximately 6 1/2 miles along U. S. Highway 141 to U. S. Coast and Geodetic Survey triangulation Station DENMARK for which the following data are known:

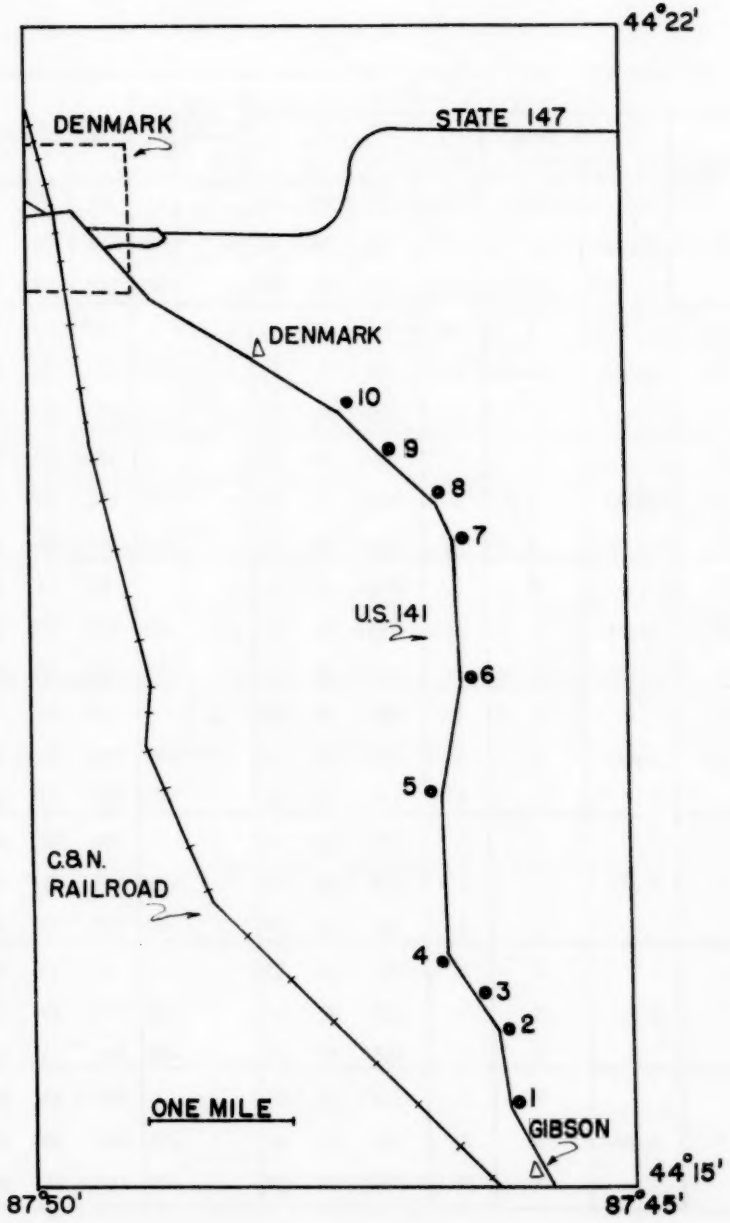
44°22'
DENMARK
STATE 147
DENMARK
10
9
8
7
U.S. 141
6
5
C.&N.
RAILROAD
4
3
2
1
ONE MILE
GIBSON
44°15'
87°50'
87°45'

FIG. 1-12

Preliminary Azimuths							Correction		Final Azimuths		
Sta.		From Sta.	To Sta.	Azimuth o	'	"	'	"	o	'	"
Gibson	Angle	Gibson	Mk.	337	59	24			337	59	24
		Mk.	1	171	20	13		+01	171	20	14
		Gibson	1	149	19	37		+01	149	19	38
1	Angle	1	Gibson	329	19	37			329	19	38
		Gibson	2	200	51	11		+01	200	51	12
		1	2	170	10	48		+02	170	10	50
2	Angle	2	1	350	10	48			350	10	50
		1	3	154	10	52		+01	154	10	53
		2	3	144	21	40		+03	144	21	43
3	Angle	3	2	324	21	40			324	21	43
		2	4	159	30	25		+02	159	30	27
		3	4	123	52	05		+05	123	52	10
4	Angle	4	3	303	52	05			303	52	10
		3	5	234	12	17		+01	234	12	18
		4	5	178	04	22		+06	178	04	28
5	Angle	5	4	358	04	22			358	04	28
		4	6	195	01	07		+01	195	01	08
		5	6	193	05	29		+07	193	05	36
6	Angle	6	5	13	05	29			13	05	36
		5	7	153	27	49		+01	153	27	50
		6	7	166	33	18		+08	166	33	26
7	Angle	7	6	346	33	18			346	33	26
		6	8	159	50	40		+01	159	50	41
		7	8	146	23	58		+09	146	24	07

Preliminary Azimuths							Correction	Final Azimuths		
Sta.		From Sta.	To Sta.	Azimuth °	'	"	' "	°	'	"
		8	7	326	23	58		236	24	07
8	Angle	7	9	157	16	12	+01	157	16	13
		8	9	123	40	10	+10	123	40	20
		9	8	303	40	10		303	40	20
9	Angle	8	10	189	31	17	+01	189	31	18
		9	10	133	11	27	+11	133	11	38
		10	9	313	11	27		313	11	38
10	Angle	9	11	196	50	54	+01	196	50	55
		10	Denmark	150	02	21	+12	150	02	33
		Denmark	10	330	02	21		330	02	33
Denmark	Angle	10	Mk	143	50	58	+01	143	50	59
		Denmark	Mk	113	53	19	+13	113	53	32

Fig. 1-13 Computation of Grid Azimuths

Line From Sta.	To Sta.	Corrected Field Distance	Elevation Factor (a)	Scale Factor (b)	Grid Factor (a) x (b)	Grid Distance
Gibson	1	2641.78	0.9999646	1.0000439	1.0000085	2641.80
1	2	2753.14		1.0000457	1.0000103	2753.17
2	3	1498.29		1.0000468	1.0000114	1498.31
3	4	1701.15		1.0000475	1.0000121	1701.17
4	5	6305.55		1.0000512	1.0000158	6305.65
5	6	4151.11		1.0000551	1.0000196	4151.19
6	7	5175.81		1.0000581	1.0000227	5175.93
7	8	1648.20		1.0000604	1.0000250	1648.24
8	9	2478.12		1.0000616	1.0000262	2478.18
9	10	2247.55		1.0000627	1.0000273	2247.61
10	Denmark	3010.72	0.9999646	1.0000643	1.0000289	3010.81

Fig. 1-14 Computation of Grid Distances

Plane coordinates (Wisconsin-South Zone):

x = 2,575,313.71 ft.
y = 858,140.66 ft.

Grid azimuth (from south) to Azimuth mark = 113° 53' 32"

In the subsequent computation of the traverse the preceding plane coordinate positions and grid azimuths are held fixed. Fig. 1-15 shows the manner of beginning the traverse at triangulation station GIBSON and portrays the nature of the angle-to-right which was measured at all transit points.

Computation of Grid Coordinates

The computation of the traverse comprises six basic operations which are as follows: (a) adjustment of the observed angles; (b) computation of corrected grid azimuths for all traverse courses; (c) reduction of the corrected field distances (ground distances) to grid distances; (d) computation of latitudes and departures; (e) computation of preliminary grid coordinates of the traverse points; and (f) computation of adjusted grid coordinates.

a) Adjustment of the observed angles—the original field angles shown in Fig. 1-13 were obtained by repeating the angle-to-right with an optical transit reading to the nearest 0.1 minute. The resulting decimal fractions were

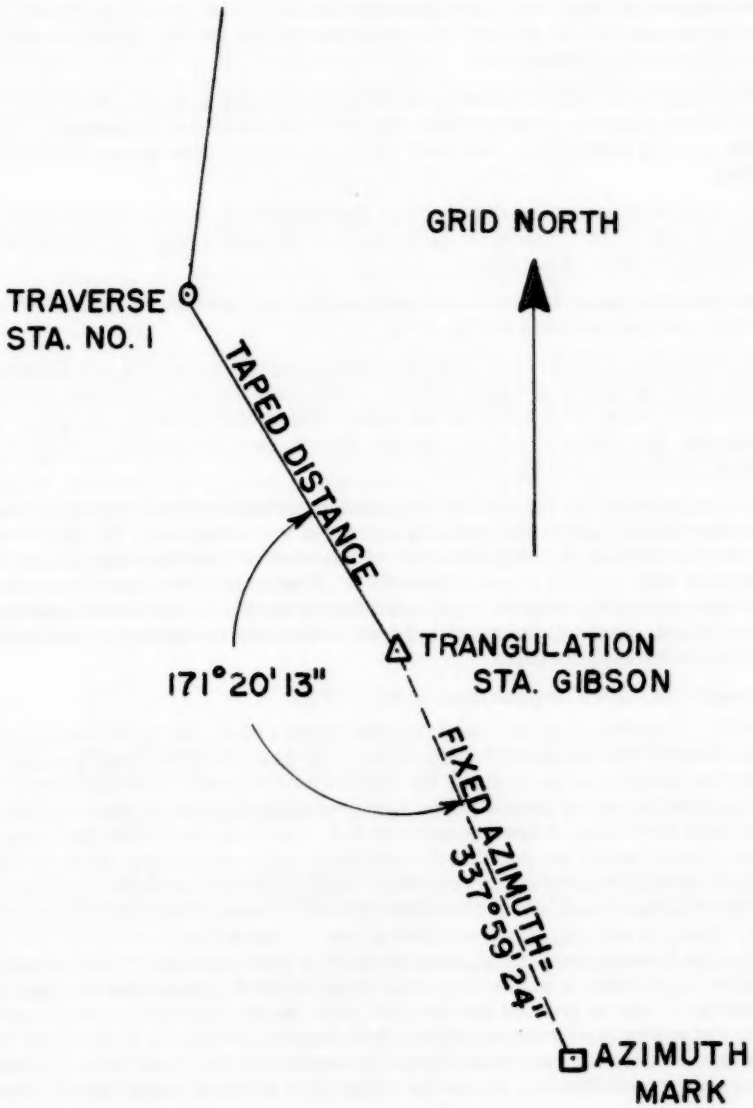
GRID NORTH
TRAVERSE
STA. NO. 1
TAPED DISTANCE
171°20'13"
TRANGULATION
STA. GIBSON
FIXED AZIMUTH = 337°59'24"
AZIMUTH
MARK

FIG. 1-15

subsequently converted to seconds for ease in using available tables of natural sine and cosine functions. The aximuth closure of 13" was distributed by assigning corrections of plus 1" to all angles except that at station 3 where a correction of 2" was applied.

b) Computation of corrected grid azimuths—these also are shown in Fig. 1-13 where it can be observed that the corrections to the preliminary azimuths are the cumulative sums of the corrections to the measured field angles.

c) Reduction of ground distances to grid distances—these computations are shown in Fig. 1-14. The scale factors were obtained from Fig. 1-6 in the manner previously described.

d) Computation of latitudes and departures—the machine computation of these quantities is shown in Fig. 1-16.

e) Computation of preliminary grid coordinates—these also are indicated in Fig. 1-16 as the first values of the coordinates for each traverse station. From the closure of +1.67 ft. in the x-coordinate and -2.43 ft. in the y-coordinate, the linear error of closure, 2.95 ft., and the relative error of closure, 1:11,400 were calculated.

f) Computation of adjusted or final grid coordinates—the familiar Compass Rule was used to adjust the preliminary x and y coordinates. The x-correction was -0.0495 ft. per thousand feet of cumulative traverse length and the y-correction was +0.0722 ft. per thousand ft. The cumulative lengths of the traverse courses are indicated in parentheses in the column of grid distances and the coordinate corrections resulting from slide rule computation are entered in the grid coordinate columns.

Comments on Grid Computations

In the computation of precise traverse which either forms the basic horizontal control for subsequent projection to the ground of the final highway centerline location or is in itself the centerline traverse, it should be emphasized that inherent anomalies will be revealed when the difference between the ground stationing of two consecutive P.I.'s is compared with the distance between those points as calculated from their grid coordinates. Such incompatibilities will be greatest in regions of high relief and in those areas of the state coordinate zones where the scale factor is most greatly different from unity. This, of course, poses a problem and is particularly vexing if the reason for the descrepancy is not understood. The best practice of dealing with this situation is probably a matter of policy to be decided by each engineering organization. It can be pointed out that for large bridge surveys and other important and complex construction layouts it is merely necessary to multiply the grid distance between two points by the reciprocal of the grid factor to obtain the correct ground distance needed to establish a desired linear layout dimension.

Attention should be directed, also, to the erroneous practice of prorating between two points of fixed state coordinate position the descrepancy revealed in closing a traverse that has been computed with ground distances. This practice is particularly subject to criticism when the traverse is long and substantial changes in scale factor are ignored, and when the changes in elevation between successive traverse courses are very pronounced. Final adjustment by prorating obviously assumes the corrections to all course lengths for scale factor and elevation to be merely functions of the measured field distance.

This section on plane coordinate computation does not attempt the explanation of a variety of special situations involving state coordinates. These include the calculation of the grid coordinates of the point of intersection of two lines defined in terms of grid position and grid azimuth, special curve calculations, and unusual traverse ties to points of known position such as through the solution of the three-point problem.

7. Interconversion of Plane Coordinates

The increasing use of state plane coordinates has led to situations in which it has become necessary to convert plane rectangular coordinates on an arbitrary or strictly local system to the state coordinate system. The circumstances under which this problem originates may be illustrated by the existence of the accurately determined coordinates of many points in a city survey system. As the municipality increases in size, and connections are made to the state coordinate system it may become desirable to place all the old data, presumed to be of good quality, on a single coordinate system, viz, the appropriate state system.

The mechanics of accomplishing this transformation are not difficult. The principles involved are equally applicable to the problem of converting the state coordinates of a point in the marginal area common to both zones of the same state from one zone to another or of effecting the conversion from Lambert coordinates to Transverse Mercator coordinates, or vice versa, in state boundary regions. For the detailed explanation of the solution of these and related problems reference is made to standard surveying texts, the technical periodical literature, and various government manuals.

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2. Kissam, Philip, Surveying for Civil Engineers, McGraw Hill Book Company, 1956.
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4. Rayner, W. H., and Schmidt, Milton O., Surveying—Elementary and Advanced, D. Van Nostrand Co., 1957.

Station	Grid Bearing ° ' "	Grid Distance (ft.)	Sin Bearing / Cos Bearing	Dept.	Lat.	Grid Coordinates x	Grid Coordinates y
Gibson					(Fixed)	2,586,009.52	828,714.15
	N.30 40 22 W.	2641.80	0.51013433	-1347.67			
		(2642)	0.86009474		+2272.20	2,584,661.85	830,986.35
						-0.13	+0.19
1						2,584,661.72	830,986.54
	N. 9 49 10 W.	2753.17	0.17054391	-469.54			
		(5395)	0.98535008		+2712.84	2,584,192.31	833,699.19
						-0.27	+0.39
2						2,584,192.04	833,699.58
	N.35 38 17 W.	1498.31	0.58266290	-873.01			
		(6893)	0.81271394		+1217.70	2,583,319.30	834,916.89
						-0.34	+0.50
3						2,583,318.96	834,917.39
	N.56 07 50 W.	1701.17	0.83030961	-1412.50			
		(8594)	0.55730239		+948.07	2,581,906.80	835,864.96
						-0.42	+0.62
4						2,581,906.38	835,865.58
	N. 1 55 32 W.	6305.65	0.03360096	-211.88			
		(14,900)	0.99943533		+6302.09	2,581,694.92	842,167.05
						-0.74	+1.07
5						2,581,694.18	842,168.12

	N.13 05 36 E.	4151.19 (19,051)	0.22653798 0.97400233	+940.40	+4043.27	2,582,635.32 -0.94	846,210.32 +1.37
6						2,582,634.38	846,211.69
	N.13 26 34W.	5175.93 (24,227)	0.23247413 0.97260258	-1203.27	+5034.12	2,581,432.05 -1.20	851,244.44 +1.75
7						2,581,430.85	851,246.19
	N.33 35 53 W.	1648.24 (25,865)	0.55336328 0.83294002	-912.07	+1372.88	2,580,519.98 -1.28	852,617.32 +1.86
8						2,580,518.70	852,619.18
	N.56 19 40 W.	2478.18 (28,354)	0.83222302 0.55444102	-2062.40	+1374.00	2,578,457.58 -1.40	853,991.32 +2.04
9						2,578,456.18	853,993.36
	N.46 48 22 W.	2247.61 (30,601)	0.72904164 0.68446935	-1638.60	+1538.42	2,576,818.98 -1.52	855,529.74 +2.20
10						2,576,817.46	855,531.94
	N.29 57 37 W.	3010.81 (33,612)	0.49939948 0.86637184	-1503.60	+2608.49	2,575,315.38 -1.67	858,138.23 +2.43
Denmark					(Fixed)	2,575,313.71	858,140.66

Linear error of closure $= \sqrt{(1.67)^2 + (2.43)^2} = 2.95$ ft.

Relative error of closure $= \frac{1}{11,400}$

x - correction $= \frac{-1.67}{33,612} = -0.0495$ ft. per 1000 ft.

y - correction $= \frac{+2.43}{33.612} = +0.0722$ ft. per 1000 ft.

Fig. 1-16

Journal of the
SURVEYING AND MAPPING DIVISION
Proceedings of the American Society of Civil Engineers

NEW HIGHWAYS AND PHOTOGRAMMETRY[a]

John G. Ladd[b]
(Proc. Paper 1307)

1. It has not been so long ago that Highway or Railroad engineers had to personally visit every mile or every foot of a proposed route in the selection of, first, the preliminary route and later the exact site of a new or relocated road, highway, or railroad—surveyors had to go out over the route and establish an extensive network of geodetic control, to be followed by topographic survey parties, to laboriously make a "plane-table" topographic map of the route. Maybe, if the terrain was easily accessible and fairly flat a topo party could map four linear miles a day. Many, many, months and even years would be required to survey and "plane-table" map the entire route, and only then could the design engineers get to work and really plan the new highway and intelligently proceed with bridges, culverts, and the like, based on the drainage pattern as shown by the contoured map.

2. Shortly before World War II, two major developments scientifically came to light—the precision aerial camera with high distortion-free lens, and photogrammetric plotting equipment designed to scientifically use this new and improved aerial photography. The War brought about an accelerated use of, and resultant improvement in both of these new developments, and proved to the world that a new era of precise high-speed topographic mapping was here to stay; it proved that the old methods were inefficient and unnecessarily time-consuming and definitely obsolete.

3. Today, the science of photogrammetry has advanced to the point where it is not only an important tool in the field of mapping, it is an essential must. This is equally true in the fields of highway location and design; photogrammetry also aids materially in various allied endeavors, such as the geological interpretation in regard to soils, rock, drainage, excavation and fill requirements, location of borrow pits etc. The photogrammetrist is therefore an important and even an essential partner as a consultant to the highway engineer in the present stupendous national highway development and construction program. Many, and perhaps the majority of the State and County

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a. Presented to the American Society of Civil Engineers, National Convention, Jackson, Miss., February 19, 1957.

b. Col. (Retired), U. S. Army; Executive Secy., Assn. of Professional Photogrammetrists, Washington, D. C.

Highway Departments, are using intensively the photogrammetric engineer as a producer and consultant in their extensified program of implementing this national program, as well as in their local requirements. Yet, at the same time, there are a few who have not yet realized the potentials in savings, in time and dollars (not to mention greater accuracy) through the proper and complete use of photogrammetry. The use of the photogrammetric engineer as a consultant in the implementation of their programs is today an economic must. Let me enumerate a few of the important services that are presently available in this field.

4. First, the photogrammetric consultant can provide in a relatively few days, aerial photos and photo-mosaics of a proposed route, at low cost, without the necessity of a highway engineer going personally into the field (and all this without the immediate knowledge of the right-of-way and abutting property owners). From this mosaic picture of the terrain in question one can, in a matter of days, make a preliminary route selection. Shortly after this initial selection the photogrammetric engineer can provide a detailed large-scale, topographic, contoured compilation or strip map, of the final route, using either the original or new low-level aerial photography, from which, based on the drainage pattern shown by the contouring, can be made the final layout and detailed design—all in a matter of a few weeks, and without the necessity of personally traversing the terrain, or even disclosing to the landowner and real estate exploiter the knowledge of your intent. Next, your photogrammetric engineer and consultant can furnish, through photo interpretation, a complete analysis of the terrain, special information on the drainage pattern, soils evaluation, etc. This information will assist in bridge and culvert location and design, under and overpasses, and interchanges, or whatever one wants to call them.

I am not speaking about theory—I speak of proven facts. These things not only can be done, but are being done, on a consultant basis. There are dozens of proven cases, all of which show greater efficiency through higher accuracy, appreciably quicker and with material reductions in final cost.

Photogrammetric engineers, through experience and advanced knowledge of the modern equipment have now found ways to extend geodetic control so that much less ground-established control is necessary. This in itself represents a great saving of time and money, for time is money.

5. Probably the most essential contribution of the photogrammetrist as a consultant is his professional integrity. Yes, machines are nearly human, yet there is no substitute for the human equation. There are many, many types and sizes of photogrammetric instruments and equipment, some made in Europe, and some manufactured here in the United States. It is sometimes said that because I or my company has X type of equipment, that only my company can and should do the work. My answer is "Balony!"; it is, in the final analysis, the man behind the equipment that counts—the professional know-how and integrity of the individuals or company offering to do the job—any representative of a reliable company when he says he can do the job to your specifications, through his technical ability and equipment, should be trusted. He knows how to get the necessary results from the equipment even though it may be of a different manufacturer than that of his competitor. There is no major photogrammetric equipment currently manufactured today (domestic or foreign) that cannot, with proper technical and professional operators, turn out large-scale, topographic map compilations, satisfactory to meet all United

States highway requirements. The professional and operational staffs of each company, although they have different personal characteristics, have, within limits, their own way or ways to solve the operational requirement through use of their individual equipment. In other words, "there is more ways than one to skin a cat."

Don't therefore be swayed abnormally by evaluation through equipment only, because it is not the answer. Confidence in the established reputation and proven integrity of a company or an individual is the best answer.

6. Now, after having generalized the consultant's position as an aid to highway location and design, let's look for a minute at the situation in more detail.

The photogrammetric engineer as a consultant can furnish in order the following:

First, a reconnaissance survey of a general area to determine the most desired and feasible routes. (This may be a map or a mosaic.)

Second, a similar reconnaissance survey of alternate routes, and a determination of the best route topographically (disregarding politics and economics).

Third, the preliminary survey of the final route, and preparations for final design and construction plans. This calls for the smallest contour interval and the largest scale in industrial and highly populated areas, and the reverse (within limit) where the opposite is true. (This means mountainous areas and where land-use or interest is small.)

Fourth, the final location survey for the staking on the ground of the proposed highway in preparation for actual construction.

Fifth, I suppose this is the paying of toll to roll along at 60 to 65 miles an hour in perfect comfort, except boredom.

7. Now in providing the first four of the above essentials, the photogrammetric engineer can in detail furnish the individual photographs of the terrain, or rather the route in question, as a single print (or prints), as stereoscopic pairs, and as mosaics, uncontrolled, semi-controlled, and/or controlled; each have their respective use in the successive steps of highway location and design. The first, uncontrolled mosaics are effective only for illustrative purposes, and the latter two, semi-controlled and controlled, as temporary substitutes for the final contoured map compilation. Each step adds a higher degree of accuracy in horizontal measurements. The contoured products add the vertical features so essential to final layout and design.

The stereoscopic pairs are the essential in providing photo-interpretation, as well as providing through the photogrammetric equipment the vital measurements, etc. for the contoured compilation. It is the photogrammetric consultant that solves the problem of getting out of these stereo models the vital data, qualitative and quantitative, so critically needed by the highway engineer. Working as a team they can perform the engineering steps, stage by stage, required in location and design. One furnishes the basic information and the other the use, or implementation.

8. A word about accuracies—modern photogrammetric equipment combined with technical know-how can and does provide accuracies sufficient to meet, in accordance with the various scales, the standards of "National Map Accuracies." These are as established by the United States Government Mapping and Charting agencies, and are equal to or in excess to those required for highway work.

9. The highway engineer and the photogrammetric engineer must work as a team. One is the providor of the raw and finished material and the other is the user, each must of necessity know the problems of the other. The advantages and disadvantages, as well as limitations of photogrammetry should be explained to the highway engineer and in turn the latter should coordinate with the former all information available concerning the need and requirements of location and design engineering. They, in turn, can then gear their operations accordingly.

Specifications should be prepared so as to clearly spell out the required end products, at specified ranges of accuracies. The in-between details should be left to the photogrammetric engineer as a professional responsibility. Photogrammetry as a professional science is fully capable of furnishing the how, when told what!!! The highway engineer is a professional man, and so is the photogrammetric engineer. Mutual respect and trust is essential to good results, economy, and efficiency.

10. Specific contribution of the photogrammetrist as a consultant

a. Aerial photographs furnish vital information concerning the highway route, such as topography, soils, rocks, vegetation, drainage, and land use.

b. Semi- and controlled mosaics and photogrammetric compilations furnish the necessary dimensional data, such as area, shape, direction, and position.

c. Photogrammetric compilations or maps make possible the acceleration of layout and design problems.

d. The inaccessible terrain (either geographic or political) is made quickly accessible. Time required in going through swamps and/or rugged areas for ground inspection and surveys is avoided, and information revealed by ground survey (in the early stages) is eliminated and land exploitation curtailed.

e. Photo mosaics and/or the compilations, eliminate the on-the-ground reconnaissance and the routine destruction of vegetation, trees, etc. along the proposed routes, needed to provide profile and cross-section data.

f. The consultant's products can provide data essential to long-range planning, such as acquiring right-of-way property before speculation and profiteering can have its effect.

g. The photographs and the consultants can provide the allied activities grouped with essential data, such as location for adjoining parks, landscaping, traffic, soils, drainage, and availability of construction material (quarries, sand, and gravel).

h. The photo, once acquired, and other subsequent products, enable all subsequent work, except final staking and construction, to be carried forward indoors, winter or summer, rain or shine, night or day.

i. The mosaics and compilations provide a picture of the entire project as a whole as well as detailed study of selected parts.

j. The contoured compilation, provided by the consultant, furnishes the essential dimensional data for computing cut and fill, now so effectively furnished by electronic computers.

k. Photogrammetric products provide more accurate results than those formerly obtained by ground survey and mapping methods, and accomplishes the work so many times quicker and cheaper.

I have tried only to hit the high spots in these advantages in using photogrammetry and its experts as consultants. There are many other items of importance but possibly of lesser value, which time does not permit me to present here.

11. In conclusion, important quotes are in order from key Federal, State Highway and Turnpike officials—

a. As early as September 1955, Mr. Rex Fulton, Senior Highway Engineer, California Division of Highways, said, "Photogrammetry has a distinct advantage over ground survey by providing wider coverage and enabling the engineer to see possibilities which otherwise might be overlooked," and "We have discontinued the use of plane-table maps in competition with photogrammetry as we have found the plane-table (map) to be much less accurate." He further stated, "photogrammetry, especially in large-scale mapping, has developed rapidly since World War II, bringing substantial savings in cost, time, and man-power."

b. Mr. William Pryor, U. S. Bureau of Public Roads, states, "The use of aerial photography is the only economical way of establishing a road location."

c. Mr. L. L. Funk, California Division of Highways, just last month stated as follows: "The use of reconnaissance mapping has practically eliminated the necessity of making preliminary field surveys of several alternate routes in order to determine the best location," and "after the route of the highway has been determined within rather close limits. . . . the next step is to obtain large-scale, small contour interval photogrammetric mapping for detailed design of the freeway facility." And, "The use of large-scale photogrammetric mapping has been rapidly increasing in the past five years and is now generally accepted as standard practice for the design of freeways in California."

d. Mr. G. B. Gilbert, Assistant to the Chairman, Pennsylvania Turnpike Commission, says, and I quote "Aerial photogrammetric methods are:

"1. dependable
2. faster
3. more complete
4. cost less
5. avoid premature disclosure of routes."

And that's the situation in a nut-shell—the services available by the photogrammetric consultant to expedite highway location and design are in keeping with the tempo of the times and the National Highway Program—speedy, accurate, efficient, and economical.

Journal of the
SURVEYING AND MAPPING DIVISION
Proceedings of the American Society of Civil Engineers

CONTENTS

DISCUSSION
(Proc. Paper 1310)

Page

An Industry Puts the Texas Coordinate System to Work, by H. P. Cooper. (Proc. Paper 580. Prior discussion: 730. Discussion closed.) ---*

A New Look at the Specifications for Federal Geodetic Control, by Robert H. Randall, (Proc. Paper 722. Prior discussion: 845. Discussion closed.) ---*

Coordinated Surveying and Mapping for Industry, by E. D. Morse. (Proc. Paper 1064. Prior discussion: none. Discussion closed.)
by Staley W. Mims 1310-3
by Virgil A. Walston 1310-4

Experience of the Bureau of Public Roads in Highway Surveys, by William T. Pryor. (Proc. Paper 1117. Prior discussion: none. Discussion closed.)
Corrections 1310-7

Note: Paper 1310 is part of the copyrighted Journal of the Surveying and Mapping Division of the American Society of Civil Engineers, Vol. 83, SU 1, July, 1957.

* There will be no closure.

Discussion of
"COORDINATED SURVEYING AND MAPPING FOR INDUSTRY"

by E. D. Morse
(Proc. Paper 1064)

STALEY W. MIMS,* M. ASCE.—In preliminary correspondence with the author and a study of his paper it was found that a printing mistake occurred in the handling of two papers, one submitted routinely that covered an oral presentation at an ASCE meeting and the other, a subsequent revision of the paper for publication. The mistake had to do with the accompanying illustrations and the corrections for the published paper (1064) are: Fig's. 3 and 4 as printed should be changed to Fig's. 1 and 2, respectively, and all other figures printed should be deleted.

In the synopsis the comment on the apparently lethargic attitude of practitioners toward the acceptance and utilization of the State Coordinate System is well taken. The press of immediate and concurrent economic factors particularly in the initial and early days of any organization and the uncertainties of that venture frequently cause minimum essentials to become the criterion of practices. Use of any standard of production over a period of time tends to fix and perpetuate these criteria and develops serious obstacles to change for the better. This gives emphasis to the value of this paper in that it points out that actual important achievements under such circumstances may be accomplished and also shows that established criteria of production standards can be changed with economic justification.

Many surveyors, of the writer's knowledge in this section of the Nation, are confronted with the same basic problems as those outlined in the early surveying and mapping problems of the Houston utility company. These problems become more and more worrisome as the tempo of a program is accelerated. The early landmarks, so essential and important in the locative and descriptive elements of property surveying and mapping in the Spanish Influence Areas, are fast disappearing. In general the early surveying and mapping was totally without coordination except for very limited extent, primarily that of adjoiner references. Historic titles of land, its records, and other pertinent matters necessitate the following of certain historic procedures by present day practitioners of surveying and mapping. The inclusion of extensive supplemental efforts to convert totally from original standards and methods of surveying to greater precision and providing for future dependable and reliable methods of exact reestablishment appear inconsistent to many practitioners. The author notes that such common practice and prevalent attitude gives only momentary and niggardly value to much of the surveying and mapping services that are presently rendered. He does not stop at noting the condition but proceeds soundly to emphasizeing the means of correction.

With reference to the advantages of using this system the following

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* County Surveyor, Houston County and Member, Texas State Board of Registration for Public Surveyors, Tex.

extension from a slightly different viewpoint is tendered. The engineering, surveying, and mapping activities performed throughout any area by the many groups requiring such service is of significant economic importance to the ones making the direct payment and also to the public at large. Lack of any base correlation of city, county, pipe-line, railroad and other independently prepared maps, to permit consolidation into a single compilation of value, demonstrates that the public had paid several times, indirectly, for something that they ultimately had to pay again, indirectly, to the Houston utility. This is not intended and should not be construed to be a criticism of the utility company, but raises instead the question within the professions concerned of the propriety of allowing immediacy and individual project interest to hold such high preference.

Until such time as mandatory clauses replace discretionary clauses in State Laws covering the use of State Plane Coordinate Systems in surveying and mapping, the ideal of its fullest use will depend upon the dissemination of information and knowledge of actual cases of its use. The author assists materially in this objective and is surely persuasive to all concerning its benefits.

This paper brings to sharp focus the value of many diverse groups pooling their resources and activities to the end that all may benefit by obtaining more economic and efficient production standards and the rendering of a better professional service.

In the conclusion the author properly makes the point that the utilization of the State Plane Coordinate System offers a real opportunity to prove the old adage, "The true worth of a thing is not what it costs but what you get for your money."

VIRGIL A. WALSTON,[1] M. ASCE.—The author's factual presentation of an interesting case history, covering the gradual development of a coordinated control network by a large public utility company over a period of approximately twenty years,[2] projects several points for discussion by civil engineers and land surveyors serving the oil industry. The long-term economies, as well as the numerous day-to-day advantages, to be realized from such a network are unquestioned for any company with similar surveying and mapping problems. The value of full utilization of available U. S. Coast & Geodetic Survey triangulation data is also clearly demonstrated.

However, the paper is somewhat provocative, in attributing to industry as a whole, a reluctance to use State Plane Coordinates in their surveying and mapping programs because of a mistaken idea of the complexity and cost involved relative to the lasting benefits that can be derived.[3] As to their complexity, a large number of companies have the equipment and the trained engineers and surveyors who are capable of executing a coordinated control network from existing or new triangulation. The cost of such work is also known, at least in general terms.

Perhaps it is true that not all segments of industry are thoroughly familiar with the cumulative advantages over a period of years which, in many instances, will result from a coordinated control network. However, the magnitude of the economic benefits to be derived by a particular company depends,

1. Div. Civ. Engr., Humble Oil & Refining Co., Houston, Tex.
2. Synopsis of Paper 1064.
3. See first sentence in Introduction of Paper 1064.

in the final analysis, upon the nature of the surveying and mapping problem of that company. The writer is more familiar with the surveying and mapping problems that arise in the exploration and production phases of the oil industry, and this discussion will be concerned primarily with those problems.

Until the early thirties the oil industry was faced with the same serious mapping problem experienced by the utility company and so ably discussed by the author.[4] The principal difference of the mapping problem in question was that the oil industry required dependable maps covering thousands of square miles in many states of the Union whereas the utility company required maps of unusual accuracy in a limited area. Present day maps used by the oil industry are based on many sources of information obtained from federal, state and local sources but their accuracy is largely dependent upon aerial photography and the fact that this photography is correlated with and largely controlled by existing triangulation networks established by the U. S. Coast & Geodetic Survey. Other means of control are utilized by firms supplying aerial photography but the use of the U. S. Coast & Geodetic triangulation network and local extension thereto, is by far the most effective and economical method employed. The oil industry was one of the first to recognize aerial photography as an accurate and economical means of obtaining maps covering large areas and these methods of control are being constantly expanded and improved by close cooperation with mapping firms. Naturally the degree of refinement in mapping offered by these firms depends upon the end use of the particular map. Economy in their mapping program is always a paramount issue.

Placing a land survey on a coordinate system requires a survey connection to an already established station, termed a control station, whose coordinates are known. It is also very desirable that the connection be made to more than one control station. The national triangulation net executed by the U. S. Coast & Geodetic Survey comprises thousands of control stations distributed throughout the country. In some areas, the spacing of these control stations is satisfactory for land survey purposes. In other areas, the spacing is so wide that the survey connection for remote localities would not be practicable.[5] Civil engineers and land surveyors for the oil industry usually tie their surveys in lease blocks to existing triangulation stations erected by the U. S. Coast & Geodetic Survey and other governmental agencies where these monuments are located within, or in proximity to, the area being traversed. It must be remembered that the search for productive oil and gas fields, even with best of exploration knowledge and modern technique, is a great gamble, as reflected by statistics available from that industry. For that reason economy again dictates the limiting of field survey work in unproven areas to the minimum required to locate the particular tract or unit on which a test well is to be located and the elimination by curative action of any discrepancies in description and vacant or unappropriated land to which the state or federal government has not relinquished title. This limited type survey is in sharp contrast to the accurate survey of an urban area for the purpose described by the author. It becomes readily apparent that the use of State Plane Coordinates in such a limited area would not always be feasible. No economic justification can be advanced to support the necessity of running

4. Early Mapping Efforts of a Houston Utility.
5. The State Coordinate Systems, by Hugh C. Mitchell & Lansing G. Simons, Special Publication No. 235, U. S. Dept. of Commerce.

several miles of traverse line outside the immediate area of interest to tie into an existing triangulation network when the odds of obtaining production from the test well are nine to one in favor of a dry hole. If the wildcat is non-productive, the company drilling the well usually loses interest in the area and no further survey work is required. If the test well comes in as a commercial producer, the original work of the surveyor can then be related to the nearest triangulation network and coordinates for all corners, natural and artificial, may be computed for future reference. This practice is becoming more frequent in the oil industry, especially in those areas where gas production is secured on blocks consisting of several thousands of acres. The many advantages to be derived from use of coordinates in surveying and mapping in such productive areas is beyond question.

Another instance in which plane coordinates are used extensively by the oil industry are in the location of tracts and well locations in offshore waters. The writer's employer was one of the 15 major oil companies who, in 1955, sponsored the cooperative project with the U. S. Coast & Geodetic Survey in re-establishment of an arc of second-order triangulation along the Louisiana coast and the fixing of the positions of 20 or more offshore platforms located in the Gulf of Mexico.[6] This was an essential and well-timed project as the corners of tracts leased from state and federal agencies are assigned coordinates related to the State Plane Coordinate systems. The position of wells drilled within these tracts must of necessity be designated by coordinates. Such well locations are usually made by occupying stations of existing triangulation arcs using ordinary triangulation methods or through utilization of electronic equipment.

The writer's company is also working with manufacturers of electronic computing machines to determine the feasibility of the use of such machines by surveyors in calculating the bearing and distances of the lines, the area of the tract, and the coordinates of each corner of the tract. If this method can be perfected, and the cost justified, the use of State Plane Coordinates in the oil industry will be greatly expanded.

In summary the writer wishes to emphasize that it is not the complexity of the problem of using State Plane Coordinates that has caused reluctance on the part of civil engineers and surveyors working for the oil industry to use the system. Many of them also realize the lasting benefits that can be derived from the use of coordinates in a survey and mapping program. These points are borne out by the preceding discussion on the ways and means State Plane Coordinates are employed in the surveying and mapping program of that industry. The fact remains that economic factors still rule out the use of coordinates in a large part of their surveying and mapping program where a high degree of accuracy is not required.

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6. Last paragraph of Legislative Act of 1947, Paper 1064.

Discussion of
"EXPERIENCE OF THE BUREAU OF PUBLIC ROADS
IN HIGHWAY SURVEYS"

by William T. Pryor
(Proc. Paper 1117)

CORRECTIONS.—Footnote b on page 1117-1 should be changed to "Highway Engr., Bureau of Public Roads, U. S. Dept. of Commerce, Washington, D. C."

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Board of Direction are identified by the symbols (BD). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper numbers are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1113 is identified as 1113 (HY6) which indicates that the paper is contained in the sixth issue of the Journal of the Hydraulics Division during 1956.

VOLUME 82 (1956)

JULY: 1019(ST4), 1020(ST4), 1021(ST4), 1022(ST4), 1023(ST4), 1024(ST4)[c], 1025(SM3), 1026(SM3), 1027(SM3), 1028(SM3)[c], 1029(EM3), 1030(EM3), 1031(EM3), 1032(EM3), 1033(EM3)[c].

AUGUST: 1034(HY4), 1035(HY4), 1036(HY4), 1037(HY4), 1038(HY4), 1039(HY4), 1040(HY4), 1041(HY4)[c], 1042(PO4), 1043(PO4), 1044(PO4), 1045(PO4), 1046(PO4)[c], 1047(SA4), 1048(SA4)[c], 1049(SA4), 1050(SA4), 1051(SA4), 1052(HY4), 1053(SA4).

SEPTEMBER: 1054(ST5), 1055(ST5), 1056(ST5), 1057(ST5), 1058(ST5), 1059(WW4), 1060(WW4), 1061(WW4), 1062(WW4), 1063(WW4), 1064(SU2), 1065(SU2), 1066(SU2)[c], 1067(ST5)[c], 1068(WW4)[c], 1069(WW4).

OCTOBER: 1070(EM4), 1071(EM4), 1072(EM4), 1073(EM4), 1074(HW3), 1075(HW3), 1076(HW3), 1077(HY5), 1078(SA5), 1079(SM4), 1080(SM4), 1081(SM4), 1082(HY5), 1083(SA5), 1084(SA5), 1085(SA5), 1086(PO5), 1087(SA5), 1088(SA5), 1089(SA5), 1090(HW3), 1091(EM4)[c], 1092(HY5)[c], 1093(HW3)[c], 1094(PO5)[c], 1095(SM4)[c].

NOVEMBER: 1096(ST6), 1097(ST6), 1098(ST6), 1099(ST6), 1100(ST6), 1101(ST6), 1102(IR3), 1103(IR3), 1104(IR3), 1105(IR3), 1106(ST6), 1107(ST6), 1108(ST6), 1109(AT3), 1110(AT3)[c], 1111(IR3)[c]; 1112(ST6)[c].

DECEMBER: 1113(HY6), 1114(HY6), 1115(SA6), 1116(SA6), 1117(SU3), 1118(SU3), 1119(WW5), 1120(WW5), 1121(WW5), 1122(WW5), 1123(WW5), 1124(WW5)[c], 1125(BD1)[c], 1126(SA6), 1127(SA6), 1128(WW5), 1129(SA6)[c], 1130(PO6)[c], 1131(HY6)[c], 1132(PO6), 1133(PO6), 1134(PO6), 1135(BD1).

VOLUME 83 (1957)

JANUARY: 1136(CP1), 1137(CP1), 1138(EM1), 1139(EM1), 1140(EM1), 1141(EM1), 1142(SM1), 1143(SM1), 1144(SM1), 1145(SM1), 1146(ST1), 1147(ST1), 1148(ST1), 1149(ST1), 1150(ST1), 1151(ST1), 1152(CP1)[c], 1153(HW1), 1154(EM1)[c], 1155(SM1)[c], 1156(ST1)[c], 1157(EM1), 1158(EM1), 1159(SM1), 1160(SM1), 1161(SM1).

FEBRUARY: 1162(HY1), 1163(HY1), 1164(HY1), 1165(HY1), 1166(HY1), 1167(HY1), 1168(SA1), 1169(SA1), 1170(SA1), 1171(SA1), 1172(SA1), 1173(SA1), 1174(SA1), 1175(SA1), 1176(SA1), 1177(HY1)[c], 1178(SA1), 1179(SA1), 1180(SA1), 1181(SA1), 1182(PO1), 1183(PO1), 1184(PO1), 1185(PO1)[c].

MARCH: 1186(ST2), 1187(ST2), 1188(ST2), 1189(ST2), 1190(ST2), 1191(ST2), 1192(ST2)[c], 1193(PL1), 1194(PL1), 1195(PL1).

APRIL: 1196(EM2), 1197(HY2), 1198(HY2), 1199(HY2), 1200(HY2), 1201(HY2), 1202(HY2), 1203(SA2), 1204(SM2), 1205(SM2), 1206(SM2), 1207(SM2), 1208(WW1), 1209(WW1), 1210(WW1), 1211(WW1), 1212(EM2), 1213(EM2), 1214(EM2), 1215(PO2), 1216(PO2), 1217(PO2), 1218(SA2), 1219(SA2), 1220(SA2), 1221(SA2), 1222(SA2), 1223(SA2), 1224(SA2), 1225(PO)[c], 1226(WW1)[c], 1227(SA2)[c], 1228(SM2)[c], 1229(EM2)[c], 1230(HY2)[c].

MAY: 1231(ST3), 1232(ST3), 1233(ST3), 1234(ST3), 1235(IR1), 1236(IR1), 1237(WW2), 1238(WW2), 1239(WW2), 1240(WW2), 1241(WW2), 1242(WW2), 1243(WW2), 1244(HW2), 1245(HW2), 1246(HW2), 1247(HW2), 1248(WW2), 1249(HW2), 1250(HW2), 1251(WW2), 1252(WW2), 1253(IR1), 1254(ST3), 1255(ST3), 1256(HW2), 1257(IR1)[c], 1258(HW2)[c], 1259(ST3)[c].

JUNE: 1260(HY3), 1261(HY3), 1262(HY3), 1263(HY3), 1264(HY3), 1265(HY3), 1266(HY3), 1267(PO3), 1268(PO3), 1269(SA3), 1270(SA3), 1271(SA3), 1272(SA3), 1273(SA3), 1274(SA3), 1275(SA3), 1276(SA3), 1277(HY3), 1278(HY3), 1279(PL2), 1280(PL2), 1281(PL2), 1282(SA3), 1283(HY3)[c], 1284(PO3), 1285(PO3), 1286(PO3), 1287(PO3)[c], 1288(SA3)[c].

JULY: 1289(SM3), 1290(EM3), 1291(EM3), 1292(EM3), 1293(EM3), 1294(HW3), 1295(HW3), 1296(HW3), 1297(HW3), 1298(HW3), 1299(SM3), 1300(SM3), 1301(SM3), 1302(ST4), 1303(ST4), 1304(ST4), 1305(SU1), 1306(SU1), 1307(SU1), 1308(ST4), 1309(SM3), 1310(SU1)[c], 1311(EM3)[c], 1312(ST4), 1313(ST4), 1314(ST4), 1315(ST4), 1316(ST4), 1317(ST4), 1318(ST4), 1319(SM3)[c], 1320(ST4), 1321(ST4), 1322(EM3), 1323(AT1), 1324(AT1), 1325(AT1), 1326(AT1), 1327(AT1), 1328(AT1)[c], 1329(ST4)[c].

c. Discussion of several papers, grouped by Divisions.

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